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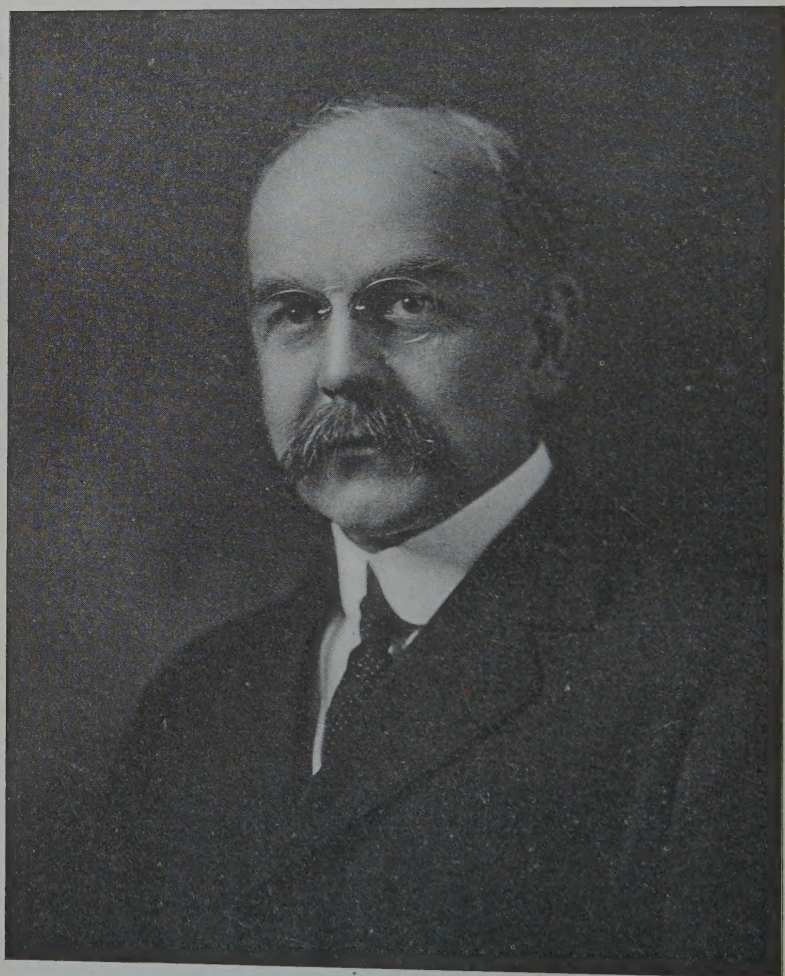
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The issue of the JOURNAL for January has been delayed, due to the closing of all manufacturing plants by order of the United States Fuel Administrator.



GEORGE C. WHIPPLE
President, Boston Society of Civil Engineers
1917-1918

BOSTON SOCIETY OF CIVIL ENGINEERS
FOUNDED 1848

PROCEEDINGS

NOTICE OF REGULAR MEETING.

A REGULAR meeting of the Boston Society of Civil Engineers will be held on

WEDNESDAY, JANUARY 23, 1918,

at 7.45 o'clock P.M., in CHIPMAN HALL, TREMONT TEMPLE, BOSTON.

Business of the Meeting.—To choose a committee of five to nominate officers for the ensuing year.

Mr. Frank B. Walker will present a paper entitled, "Loading and Hauling Iron Ore from Mesabi Range to Lake Docks." The paper will be illustrated with lantern slides.

S. E. TINKHAM, *Secretary*.

PAPERS IN THIS NUMBER.

"Foundations of the New Buildings of the Massachusetts Institute of Technology, Cambridge, Mass." Charles T. Main and H. E. Sawtell.

"Concrete Materials and Design of the New Buildings of the Massachusetts Institute of Technology, Cambridge, Mass." Sanford E. Thompson.

CURRENT DISCUSSION.

Paper.	Author.	Published.	Discussion Closes.
"Water Powers of New England."	Henry I. Harriman.	Dec. Feb. 10.	

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Contributors are hereby notified that proof will not be submitted to them for examination unless requested before the 10th of the month preceding the month of publication.

MINUTES OF MEETINGS.

CAMBRIDGE, December 19, 1917. — A regular meeting of the Boston Society of Civil Engineers was held this evening in the large lecture hall (Building No. 10), Massachusetts Institute of Technology, Cambridge, and was called to order by the President, George C. Whipple, at 7.45 o'clock.

There were 151 members and visitors present.

The record of the last meeting was read and approved.

The Secretary reported for the Board of Government that it had elected to membership, in the grade of member, Mr. Ashley Quincy Robinson.

The President announced the death of Henry A. Herrick, a member of the Society, which occurred on December 14, 1917, and by vote he was requested to appoint a committee to prepare a memoir. The President has named Mr. Charles T. Main as the committee.

The proposed amendment to By-Law 5, adopted at the November meeting, as printed in the November number of the JOURNAL of the Society, was adopted again as required by the By-Laws, 61 voting in favor and none against.

Before taking up the literary exercises, the President stated that arrangements had been made to exhibit the method used at the Institute for illustrating lectures, where a number of pieces of apparatus are employed. With the aid of a small railway track, several tables were pushed into the room at the speaker's desk, each of which had been properly fitted up in

adjoining laboratories in complete readiness for demonstrating the subjects to be treated at the lectures to be given the following day.

The President then called on Past President Charles T. Main, who read the first paper of the evening, entitled, "The Foundation of the New Technology Buildings." The paper was fully illustrated with diagrams and tables showing the results of borings and the records of pile driving.

The paper was discussed by Past President Desmond Fitzgerald, Mr. Lewis M. Hastings and others.

The second paper presented was prepared by Sanford E. Thompson, entitled, "The Theoretical Design of the Structures of the New Technology Buildings." In the absence of the author the paper was read by Mr. Carl A. Funk, of Mr. Thompson's office. The paper was also illustrated with lantern slides. A short discussion followed.

On motion of Mr. Fernald the thanks of the Society were voted to the Massachusetts Institute of Technology for its courtesy in throwing open its buildings and laboratories for the inspection of members, and for the use of its commodious lecture room for the evening's meeting.

Adjourned.

S. E. TINKHAM, *Secretary*.

BOSTON, December 5, 1917. — The regular meeting of the Sanitary Section of the Boston Society of Civil Engineers was held this evening, in the Society Library, Tremont Temple.

The meeting was called to order at 8 o'clock by the chairman, Frank A. Marston. The records of the November meeting were read and approved.

The first speaker of the evening, Mr. Stephen DeM. Gage, was introduced, and gave an illustrated talk on the subject of the sanitary conditions of swimming pools. The next speaker, Mr. S. K. Nason, physical director of the Brookline Municipal Gymnasium, described at length the Brookline Public Baths and the method of maintenance. Mr. Greenwood gave a brief description of the outdoor swimming pool in Gardner. Many

questions were asked by members and visitors, and brief remarks were made by Messrs. Blake, Clark, Whipple and Weston.

It was voted to extend the thanks of the Section to both Mr. Gage and Mr. Nason.

Forty-five members and visitors were present.

The meeting adjourned at 10.30.

HENRY A. VARNEY, *Acting Clerk.*

APPLICATIONS FOR MEMBERSHIP.

[January 10, 1918.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission, and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

EMERSON, JOHN MCNEIL, Brookline, Mass. (Age 30, b. Boston, Mass.) From June, 1908, to June, 1909, assistant superintendent at factory of Penn. Fireproofing Co., St. Marys, Pa., manufacturing hollow terra-cotta blocks; from 1909 to date, with Pennsylvania Tile and Cons. Co.; for past five years has been president of that company; work has consisted of advising on fire-proof construction, redesigning and erection. Refers to J. P. Gallagher, J. S. Humphrey, Mark Linenthal, James F. Monaghan and J. T. Scully.

KILLION, LOUIS JOHN, Roxbury, Mass. (Age 33, b. Boston, Mass.) Graduate of Mass. Inst. of Technology, 1905, civil engineering course. From

1905 to 1906, with Lewis F. Shoemaker & Co., Pottstown and Philadelphia, Pa., on drafting, shopwork and field erection; from 1907 to 1912, with H. P. Converse & Co., Boston, on drafting, designing, estimating and field erection; from 1913 to November, 1917, with Drake Bros. Co. as superintendent in charge of Boston plant; is now with engineering department of Monks & Johnson, on Destroyer Plant at Squantum, Mass. Refers to E. P. Bliss, N. C. Burrill, J. E. Carty, H. T. Gerrish, C. M. Spofford and G. F. Swain.

SAUER, FRED EUGENE, Jr., Everett, Mass. (Age 24, b. Everett, Mass.) Graduate of Mass. Inst. of Technology, 1914, civil engineering course, degree of B.S. During summer of 1914, rodman with N. Y., N. H. & H. R. R.; from August, 1914, to February, 1916, assistant engineer on subway construction with Public Service Comm., New York City; from February to August, 1916, structural draftsman with Post & McCord, New York City; from August, 1916, to November 1917, structural draftsman with Navy Dept., Washington, D. C.; from November, 1917, to date, assistant engineer with Metcalf & Eddy. Refers to C. F. Allen, C. B. Breed, H. P. Eddy and F. A. Marston.

STRAND, HARRY LANCASTER, Brighton, Mass. (Age 38, b. Charlestown, Mass.) Student at Mass. Inst. of Technology from 1899 to 1901, architectural engineering course. From 1901 to 1903, with C. B. Lancaster Shoe Co. on construction work; from 1903 to 1909, with National Fire Proofing Co. on plans and redesign; from 1909 to date, with Pennsylvania Tile and Construction Co. as manager, in charge of planning buildings, redesigning and sales. Refers to R. D. Bradbury, J. P. Gallagher, C. R. Gow, A. C. Holt, J. S. Humphrey, Mark Linenthal and J. T. Scully.

LIST OF MEMBERS.

CHANGES OF ADDRESS.

ASPINWALL, THOMAS.....54 Stearns Rd., Brookline, Mass.
 BEARD, CORNELIUS.....American Expeditionary Force, France.
 BOAS, BENJAMIN.....care Monks & Johnson, Box 9, Atlantic, Mass.
 BROWN, WILLIAM AUGUSTINE,
 United States Naval Training Station, Bumkin Island, Mass. Div. 4, Dec. 3.
 COWLES, LUZERNE S....15 Dwight St., Coolidge's Corner, Brookline, Mass.
 CUTTING, GEORGE W., Jr.....10 Post Office Sq., Boston, Mass.
 DE LA HAYE, ELIAS F., Jr.....30 Waverly St., Roxbury, Mass.
 DRUMMOND, Capt. WILLIAM W.,
 care W. D. Caiger, 44 Manthorne Rd., West Roxbury, Mass.
 HUBBARD, Serg. CARL P.,
 Co. D., 11th Engrs. (Rly), U. S. Expeditionary Force, France.
 HURD, STEPHEN P.....512 Cutler Bldg., Rochester, N. Y.
 MONAGHAN, Capt. JAMES P.,
 Ord. R. C., U. S. A., 1330 F St. N. W., Washington, D. C.

RICHMOND, Lieut. CARL G. E. O. T. C., Camp Lee, Petersburg, Va.
 SARGENT, FRANK C. 201 Devonshire St., Boston, Mass.
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 SMITH, SIDNEY care Hill & Ferguson, Camp Merritt, N. J.
 SUMNER, MERTON R. Augustine Mills, Wilmington, Del.
 WADE, Lieut. CLIFFORD L. Hawthorne House, Wellesley Hills, Mass.
 WARING, Major CHARLES T.,
 care Construction Division, Signal Corps, Langley Field, Hampton, Va.
 WIGGIN, Capt. THOMAS H.,
 Engr. Corps., U. S. R., care Gen. Taylor, Amer. Expeditionary Force, France.
 WOOD, CARL W. 147 Milk St., Boston, Mass.
 WOOD, Capt. LEONARD P. American Expeditionary Force, via New York.
 WORCESTER, Lieut. ROBERT J. H. 15 Lexington Rd., Concord, Mass.

DEATH.

HERRICK, HENRY A. December 14, 1917

EMPLOYMENT BUREAU.

THE Board of Government maintains an employment bureau for the Society, to be a medium for securing positions for its members and applicants for membership, and also for furnishing employees to members and others desiring men capable of filling responsible positions.

At the Society room two lists are kept on file, one of *positions available* and the other of *men available*, giving in each case detailed information in relation thereto.

MEN AVAILABLE.

416. Age 53. Graduate of Mass. Inst. of Technology, 1886. Experience consists of one year with U. S. Geological Survey on topographic work; about twenty-five years with City of Boston in sewer, engineering and public works departments, chiefly on sewers, parks, bridges and special investigations; and about two years with prominent engineering firm on studies and designs for system of sewers and surface water drains and on bridge design. Prefers position in Boston or vicinity. Salary desired, \$200 per month.

LIBRARY NOTES.

RECENT ADDITIONS TO THE LIBRARY.

U. S. Government Reports.

Abrasive Materials in 1916. Frank J. Katz.

Annual Report of Governor of Panama Canal for 1916-17.

Barytes and Barium Products in 1916. James M. Hill.

Corsicana Oil and Gas Field, Texas. George Charlton Matson and Oliver Baker Hopkins.

Fuller's Earth in 1916. Jefferson Middleton.

Geology and Ore Deposits of Ely, Nevada. Arthur C. Spencer.

Geology of Massachusetts and Rhode Island. B. K. Emerson.

Geology of Navajo Country. Herbert E. Gregory.

Hydraulic-Mining Débris in Sierra Nevada. Grove Karl Gilbert.

Irvine Oil Field, Estill County, Kentucky. Eugene Wesley Shaw.

Mineral Resources of Kantishna Region, Alaska. Stephen R. Capps.

Mineral Resources of United States, 1915: Part I, Metals. H. D. McCaskey.

Mining in Lower Copper River Basin and Prince William Sound Region, Alaska. F. H. Moffit and B. L. Johnson.

Our Mineral Supplies: Fluorspar, by Ernest F. Burchard; Petroleum, by John D. Northrop.

Petroleum Withdrawals and Restorations Affecting the Public Domain, Issued between January 16 and September 30, 1916.

Pulpwood Consumption and Wood Pulp Production, 1916. Franklin H. Smith and R. K. Helphenstine, Jr.

Shorter Contributions to General Geology, 1916. David White.

Silica in 1916. Frank J. Katz.

Spirit Leveling in Nevada, 1897 to 1916, inclusive. R. B. Marshall.

Spirit Leveling in Ohio, 1898 to 1916, inclusive. R. B. Marshall.

Standard Forms for Specifications, Tests, Reports and Methods of Sampling for Road Materials.

Water-Supply Papers 389, 390, 404, 408, 425-B, 438.

State Reports.

New York. Annual Report of Public Service Commission for First District for 1915: Vols. I, Appendix F, and III.

Texas. Tables of Cubic Contents of Levee Embankments. Arthur Alvord Stiles.

Texas. Tables of Velocity of Water in Open Channels, derived from Kutter's Formula. Arthur Alvord Stiles.

Municipal Reports.

Boston, Mass. Annual Report of City Planning Board for 1916.

Boston, Mass. Annual Report of Transit Commission for 1916-17.

Chicago, Ill. Chicago's Financial Dilemma. Bureau of Public Efficiency.

Fitchburg, Mass. Annual Report of Water Commissioners for 1916.

Northampton, Mass. Annual Reports of City Officers for 1916.

Miscellaneous.

American Society for Testing Materials: Proceedings for 1917. Gift of L. C. Wason.

Canada, Dept. of Mines: Iron Ore Occurrences in Canada, Vol. I, by E. Lindeman and L. L. Bolton; Portfolio of Maps illustrating same.

National Lumber Manufacturers Association: The One-Story Schoolhouse Idea, by Fitzherbert Leather; Before Federal Trade Commission — Brief on behalf of National Lumber Manufacturers Association.

Reservoirs for Municipal Water Supply. Alfred D. Flinn. Gift of author.

State Sanitation, Vol. II. George Chandler Whipple. Gift of author.

LIBRARY COMMITTEE.

NEW ENGINEERING WORK.

(Under this head a brief description of new engineering work contemplated or under construction will be presented each month. Engineers and contractors are requested to send descriptions of their work to the Secretary, 715 Tremont Temple, Boston, before 1st of each month.)

Commonwealth of Massachusetts.— **METROPOLITAN WATER AND SEWERAGE BOARD.** — *Water Works.* — All of the poles and towers for the transmission line between the Wachusett and Sudbury power stations have been erected, and are now ready to receive the insulators and the copper conductors.

METROPOLITAN WATER AND SEWERAGE BOARD. — *Sewerage Works.* — Work in progress: Section 98, Wellesley Extension.

METROPOLITAN PARK COMMISSION. — *Old Colony Parkway.* — Construction of a temporary bridge across the Neponset River between Boston and Quincy is in progress; also construction of double siphon for water and gas under new channel.

BOSTON TRANSIT COMMISSION. — *Dorchester Tunnel.* — The interior finish of Broadway Station is about completed.

Work is progressing on the interior finish of the station at Andrew Sq., and work is in progress on the station for transfer of passengers from the surface cars to the tunnel below.

New York, New Haven & Hartford R. R. Co. — *South Boston Cut Improvement.* — Enlargement of South Boston Cut to Boston Freight Terminal, to accommodate four tracks instead of two, and reconstruction of eleven overhead highway bridges, so as to span four tracks instead of two, progressing. Excavation by steam shovel has advanced from westerly end of cut to West Fourth Street. Bridge abutments being constructed at West Sixth Street.

BOSTON SOCIETY OF CIVIL ENGINEERS

FOUNDED 1848

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications

FOUNDATIONS OF THE NEW BUILDINGS OF THE
MASSACHUSETTS INSTITUTE OF TECHNOLOGY,
CAMBRIDGE, MASS.

BY CHARLES T. MAIN* AND H. E. SAWTELL,† MEMBERS BOSTON SOCIETY OF
CIVIL ENGINEERS.

(Presented December 19, 1917.)

THE purpose of this paper is to describe —

1st. The geological conditions found at the new site of the Massachusetts Institute of Technology and the methods used in determining the conditions.

2d. The foundations as built, and the reasons governing the types used.

3d. The pile tests, load tests and designs of portions of the foundations.

Our work on the foundations for these buildings was of a consulting nature, and all preliminary work up to and including the determination of total live and dead loads of superstructures going to the piles was performed by others.

The Stone & Webster Engineering Corporation's designs and construction work for all piling, footings, piers and walls, up to the underside of the first floor beams, were subject to our examination, supervision and approval.

* Consulting Engineer, 201 Devonshire St., Boston.

† Structural Engineer, with Charles T. Main.

NOTE. Discussion of this paper is invited, to be received by W. L. Butcher, Editor, 715 Tremont Temple, Boston, before March 10, 1918, for publication in a subsequent issue of the JOURNAL.

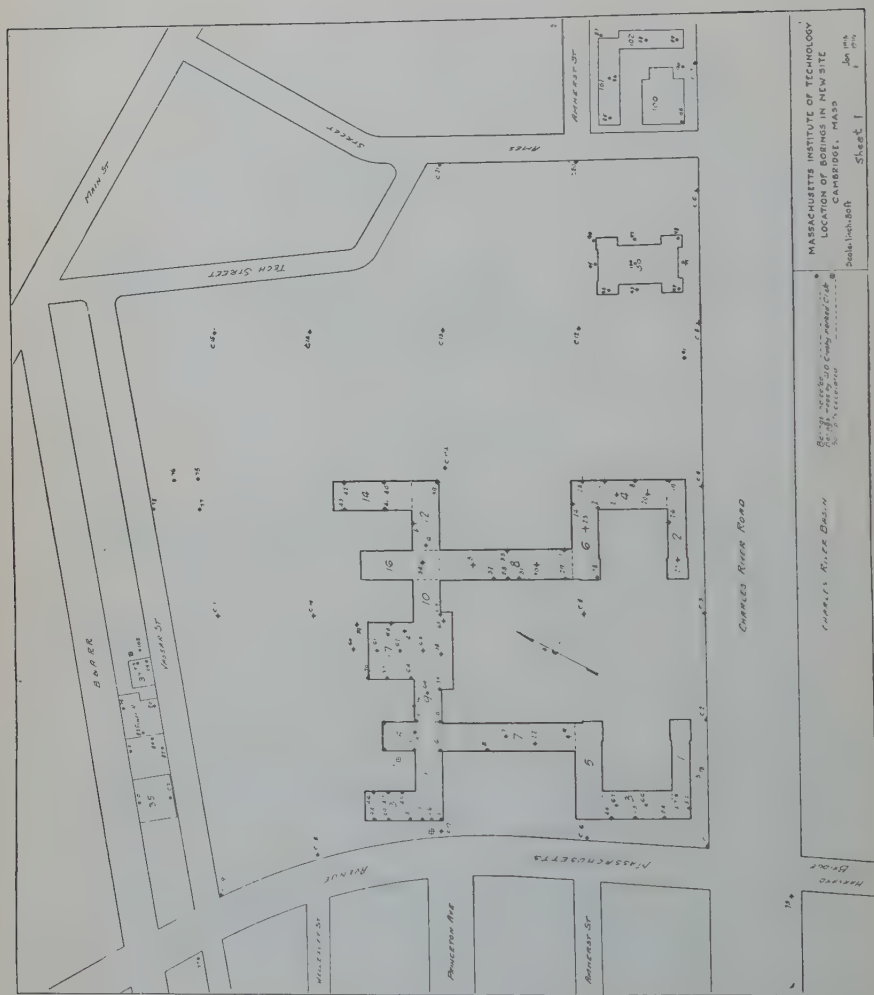
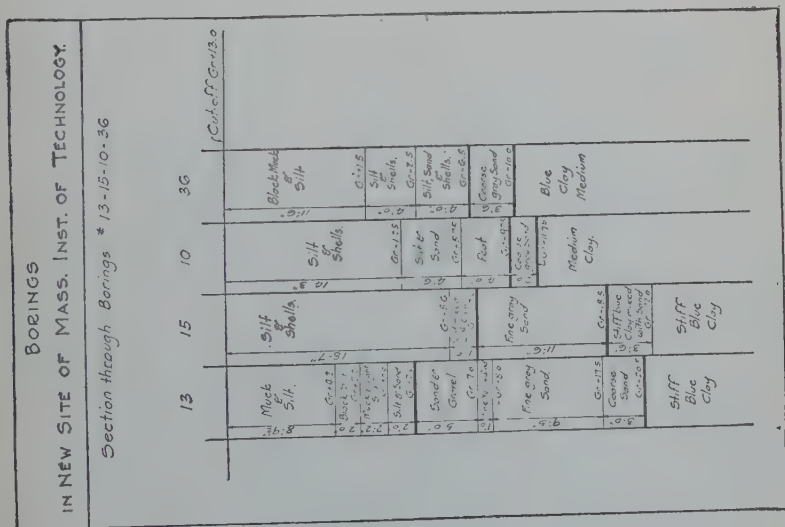
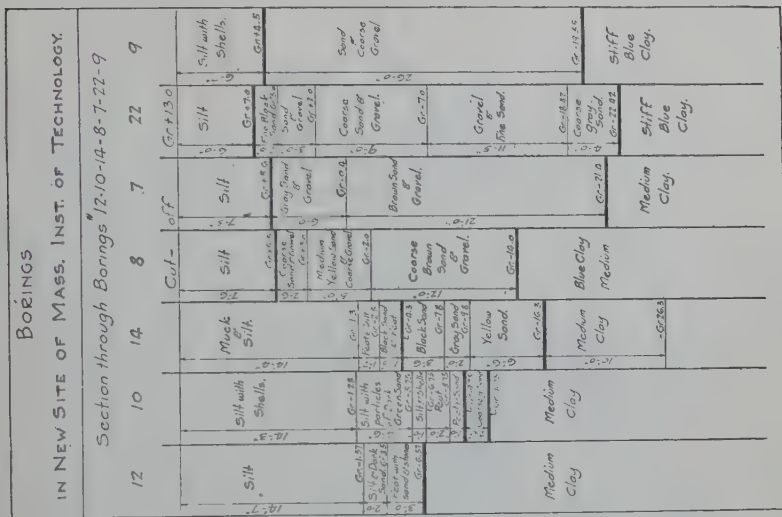


FIG. 1.



In order to design foundations for the new buildings, it was necessary to make an unusual amount of study of the soil and rock underlying the site and the supporting capacity of the same.

In July, 1913, Prof. W. O. Crosby submitted a report on the site. This report was based chiefly on the results obtained from borings in and just outside the limits of the new site.

This report shows that the bed rock is found at a depth of 120 ft. to 135 ft. from the surface, and appears to be quite level. The materials overlying the bed rock include, in succession, boulder clay varying in thickness from a few feet to 25 or 30 ft.; blue clay with a normal thickness of 60 to 100 ft.; glacial gravel ranging from nothing to 35 ft. in thickness; silt from 0.0 ft. to 18 ft. in thickness; peat, very thin and showing in a few places only, and the artificial filling at the surface varying in thickness from a few to 15 or 20 ft.

The twenty-one borings made by Professor Crosby were rather widely separated, and it was found, soon after actual work was begun, that the character and surface levels of the glacial deposit changed frequently so that it was thought necessary to make additional borings one hundred and six in number, on or near the sites of the different buildings.

The locations of both sets of borings are shown on the accompanying Fig. 1, together with some of the cross sections, Figs. 2, 3, 4, 5 and 6, which show the varying character of the upper soil which is located under some of the buildings.

The top of the glacial gravel was found to be at all levels varying from 6 ft. to 26 ft. below the cut-off grade of piles, and the thickness varied from nothing to 35 ft. In a considerable part of one or two buildings, no glacial gravel was found.

Three test pits were dug to a depth of about 22 ft., in order to check up the wash borings. Eleven concrete piles and eighty-one wooden test piles were driven, while twenty-five piles of all kinds were load tested for the purpose of further determining the character, resistance and bearing power of the substrata.

A considerable part of Professor Crosby's report is devoted to the cause of the observed settlements of structures which surround the new site. He states that it is his opinion that the

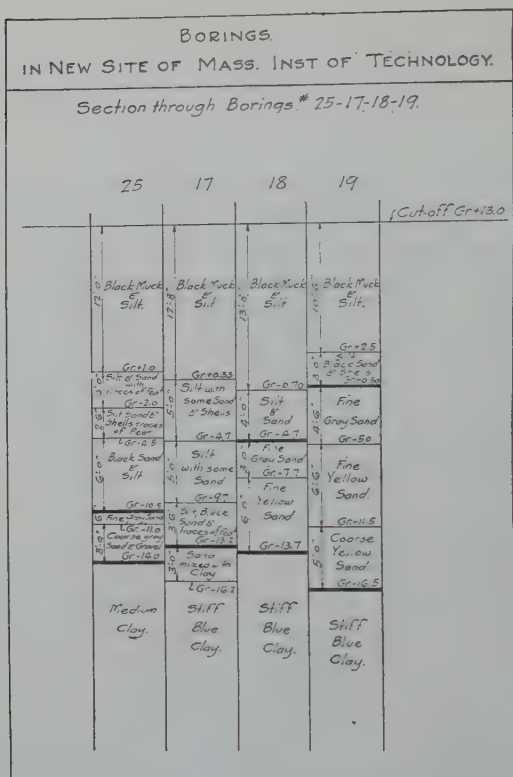


FIG. 6

settlements are only such as would be expected from a study of their foundations and the underlying earth conditions, and that it is his belief that had the foundations of these structures been carried down to or supported on the glacial gravel, they would not have settled to any appreciable extent.

The term "glacial gravel" is understood to mean that deposit immediately found above the clay, which varies in character from fine sand to coarse gravel. As it is found generally to be well compacted and comparatively near the surface, it was decided to support the foundations upon it wherever practicable.

The clay found under the glacial gravel is of remarkably

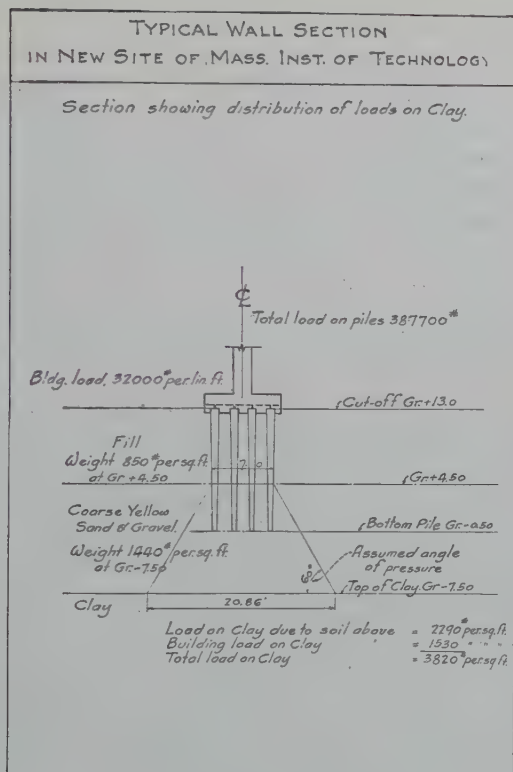


FIG. 7.

uniform character, of little plasticity, and — to use Professor Crosby's words — "It is probable that in its normal condition in the ground all of the clay is devoid of excess moisture and fairly to be described as stiff."

It has been assumed, however, that if settlement occurs, it will be in the clay, and in order to reduce this settlement to a minimum it was decided to spread the building loads over the glacial gravel as much as practicable, which in turn would still further distribute the loads over the clay bed.

In order to provide for a wide distribution of load on the gravel in the most uniform manner, with a resulting low pressure per square foot, it was decided to use a large number of

wood piles, each sustaining a relatively small load, rather than a few heavily loaded piles. See Fig. 7.

This shows the piling under a footing resting in 12 ft. of gravel or sand, and the assumed spreading of the load accomplished by means of 60 degree angles of distribution, with the horizontal. The building loads in this case were reduced to about three fourths of a ton, on the surface of the clay, which would be in addition to the load of the overburden already found on the site in place.

The decision that heavy, concentrated loads were to be avoided precluded the use of concrete piles for this particular work.

It was also found that wood piling would be considerably less expensive than concrete piling, due to the fact that the permanent water level was high, making it unnecessary to place any concrete in addition to that required for the foundations to meet the heads of the wood piling, also to the fact that large piles do not prove economical where flexibility is required for light as well as heavy loads.

Interesting tests were made, however, on concrete piles of both the Simplex and Raymond types.

Fig. 8 is a plan of the site, with the buildings in outline, showing the location of most of the important test piles; and near the location of each pile has been placed a letter, in addition to its number, to designate its kind,—“S” meaning spruce, “O” for oak, “P” for southern pine, etc.

Spruce piles from Nos. 1 to 30, inclusive, were driven from September 8, 1913, to September 25, 1913; spruce piles Nos. 31, 32, 33 and 34 were driven on December 2, 1913; oak piles 1 to 16 inclusive were driven on December 2, 3, 4 and 5, 1913; and piles in Building No. 17 were driven in September and October of 1914.

Other piles were driven and tested as noted in Tables I, 2 and 3.

The factor of safety at $\frac{1}{4}$ -in. settlement was arrived at by dividing the load carried at $\frac{1}{4}$ -in. settlement by the load carried at $\frac{1}{16}$ -in. settlement.

If the safety factor was not as high as $2\frac{1}{2}$, then the tonnage

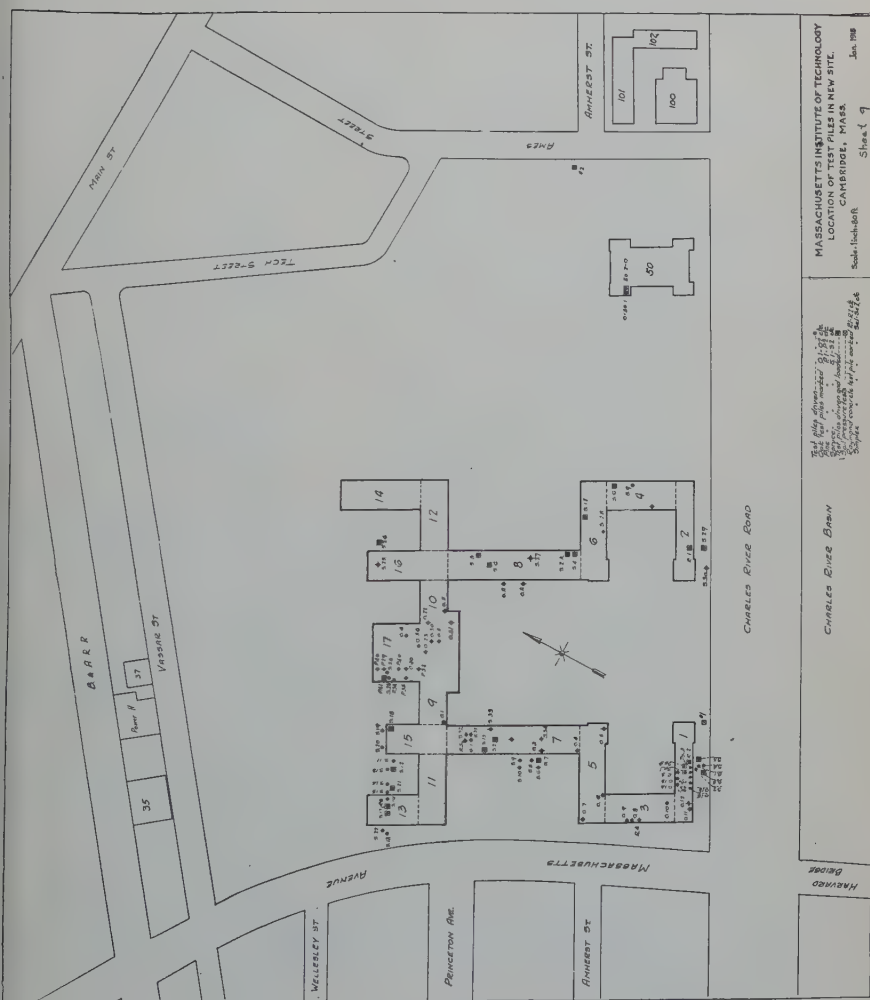


FIG. 8

TABLE I.

TEST PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY
GROUP A. Spruce.

IDENTITY.		Length of Pile.	EMBEDMENT.		SURFACE GRADES.		Ave. Drop Last Blows 2300-lb. Hammer.	Ave. Pen. Last 3 Blows.	Formula Load. Tons.	Test Load at $\frac{1}{16}$ " Settlement.
Bldg. No.	Pile No.		Sand.	Clay.	Sand.	Clay.				
Near No. 1 ↑ Near No. 7 ↓	1	31' 7"	?	0	+3.0		9' 9"	3"	...	...
	2	32' 9"	6' 6"	0	+6.25		7' 2"	$\frac{5}{16}$ "	...	...
	3	17' 0"	3' 9"	0	+4.01		9' 8"	$\frac{5}{16}$ "	16.70	6.0*
	4	20' 0"	1' 6"	0	+5.30		4' 10"	$\frac{5}{16}$ "	...	...
	5	20' 0"	5' 1"	0	+5.13		6' 2"	$\frac{5}{16}$ "	8.70	7.0*
	6	20' 0"	3' 9"	0	+6.66		9' 6"	$\frac{7}{8}$ "	...	...
	7	20' 0"	5' 9"	0	+6.29		10' 0"	1 $\frac{3}{4}$ "	8.40	3.1 tons
	8	30' 0"	8' 9"	0	+5.51		6' 0"	1"	...	...
	9	30' 0"	4' 10"	0	+4.86		11' 10"	3 $\frac{1}{4}$ "	6.50	...
	10	30' 0"	10' 11"	0	+5.23		7' 7"	$\frac{3}{4}$ "	13.90	...
Near 15	11	20' 2"	2' 8 $\frac{3}{4}$ "	0	-0.14	-12.58	8' 0"	2"	7.80	...
Near 11	12	30' 0"	12' 0"	0	-0.70	-12.00	9' 0"	2"	6.90	3.75±
Near 11	13	36' 0"	8' 0"	5.75'	-5.00	-13.0	9' 0"	1 $\frac{1}{4}$ "	9.20	...
Near 11	14	36' 0"	20.75	-0.25			5' 0"	$\frac{1}{2}$ "	...	...
Near 13	15	32' 0"	13.67	-2.67			10' 2"	2"	...	...
13	16	36' 0"	18.75'	0	+3.53	+15.50	11' 2"	2"	8.50+	...
13	17	36' 0"	15.50'	1.56'	-1.50	-17.0	10' 0"	2"	7.70	9.0*
Near 13	18	36' 0"	16.30'	1.0'	-1.25	-17.5	10' 5"	1 $\frac{1}{2}$ "	9.66	...
Near 15	19	36' 0"	0	13.43'	0	-6.57	11' 0"	5"	4.20	...
Near 15	20	45' 0"	6.0'	22.33'	-6.58	-12.5	10' 6"	3 $\frac{1}{2}$ "	...	...
Near 11	21	44' 0"	14.5'	12.5'	-1.0	-15.5	11' 0"	1 $\frac{1}{2}$ "	10.1	Emb
Near 13	22	45' 0"	12.0'	13.45'	-2.25	-14.25	10' 5"	1"	12.0	...
17	23	36' 0"	8.0'	9.37'	-3.0	-11.0	6' 4"	1 $\frac{1}{2}$ "	...	...
17	24	44' 0"	10.0'	15.20'	-2.0	-12.0	9' 2"	1 $\frac{1}{2}$ "	9.40	...
Near 16	25	45' 0"	13.25'	14.50'	+2.25	-11.0	10' 9"	1 $\frac{1}{2}$ "	14.3	...
Near 12	26	30' 0"	11.8'	1.95'	+1.40	-10.4	10' 6"	2"	8.05	11.5
Near 8	27	41' 0"	5' 9"	3' 9"	-5.75	-11.50	10' 9"	1 $\frac{5}{16}$ "	10.7	...
6	28	42' 0"	13' 6"	6' 4"	-1.50	-15.00	8' 6"	$\frac{1}{2}$ "	14.0	...
Near 2	29	30' 0"	10' 0"	0' 6"	+1.30	-8.60	6' 6"	$\frac{5}{8}$ "	9.2	9*
Near 2	30	40' 0"	13' 6"	0	-0.80	-18.0	13' 6"	$\frac{5}{8}$ "	20.0	...
7	31	25' 0"	11' 7 $\frac{1}{2}$ "	0	+3.80		7' 0"	1"	8.0	...
7	32	34' 0"	17' 10"	0	+3.10		5' 0"	1 $\frac{1}{2}$ "	7.6	...
7	33	30' 0"	10' 0±	0	+4.40		5' 0"	$\frac{1}{2}$ "	7.7	...
7	34	18' 0"	5' 4"	0	+5.60		5' 0"	1"	5.7	...

* Found by interpolation.

FOUNDATIONS OF M. I. T. NEW BUILDINGS.

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TABLE 1. — *Continued.*

TEST PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.
Spruce.

More.	CHARACTER.		Formula Values at Other Embedments.	Reported Condition when Driven.	Actual Condition when Pulled.	Remarks.
	Sand.	Clay.				
	Gravel			Broken	Broken	
	Gravel			Broomed	Broomed	
	Gravel			Good pile	Broken and Broomed	
	Gravel			Hard driving	Broken	
	Gravel			Uniform driving	Broken and broomed	
	Gravel			Broomed	Broken and broomed	
	Gravel			Uniform driving	Broken and broomed	
	Gravel			Broken		
	Gravel			Uniform driving	Broken	
	Gravel			Hard but	Broken	8" crust
	Gravel			Uniform driving		
	Sand					
	Fine sand		10 tons @ 10.5'	Uniform driving	OK	Early settlements appear excessive
			10.3 tons @ 7.0'	Uniform driving		
			8.2 tons @ 9.0'	Follower	used	
			8.0 tons @ 12.5'	Follower	used	
	Sand		8.7 tons @ 12.0'	Easy driving		
2.50	Fine sand		8.6 tons @ 12.0'	Easy driving	OK	
	Sand			Uniform driving		
	o			Easy driving		
				Follower	used	
ement	t readings	of load te	st, doubtful.			
	Sand		11.4 tons @ 12.0'			
			7.5 tons @ 6.0'	Follower	used	
			9.0 tons @ 7.0'	Uniform driving		
	Sand	Blue medium	9.2 tons @ 10.0'	4.6' of clay and	shells above	the sand.
	Sand					
	Sand		12.2 tons @ 10.0'	Uniform driving		
	Coarse	Clay and	11.2 tons @ 10.0'	Uniform driving	Tip slightly broomed	Early settlement readings in question
	Sand	Gravel			around edges	
	Sand		14.0 tons @ 10.0'	Probably broomed		
	Fine sand				Pulled OK	
	Fine sand		9.1 tons @ 11' 6"		Pulled OK	
	Fine sand		9.1 tons @ 9' 0"		OK	
	Fine sand		8.0 tons @ 1' 0"		OK	
			5.8 tons @ 2' 0"			

TABLE 2.

TEST PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

GROUP B. Oak Piles.

IDENTITY.		Length of Pile.	EMBEDMENT.		SURFACE GRADES.		Ave. Drop Last Blows 2300-lb. Hammer.	Ave. Pen. Last 3 Blows.	Formula Load. Tons.	Test Load at $\frac{1}{4}$ " Settlement.
Bldg. No.	Pile No.		Sand.	Clay.	Sand.	Clay.				
7	1	20' 6"	7' 6 $\frac{1}{2}$ "	...	4.0		8' 0"	1 $\frac{1}{4}$ "	7.1	...
7	2	20' 8"	9' 8 $\frac{3}{4}$ "	...	6.0		6' 0"	1 $\frac{1}{4}$ "	7.3	...
7	3	15' 3"	4' 10"	...	6.6		5' 0"	1 $\frac{1}{4}$ "	7.7	...
5-7	4	15' 0"	4' 11"	...	5.9		6' 0"	1 $\frac{1}{4}$ "	5.6	...
5	5	20' 9"	10' 1"	...	6.2		5' 0"	1 $\frac{1}{4}$ "	6.2	...
5	6	20' 8"	8' 7"	...	9.9		5' 0"	1 $\frac{1}{4}$ "	9.2	...
5	7	15' 0"	7' 5 $\frac{1}{2}$ "	...	9.4		5' 0"	1 $\frac{1}{4}$ "	9.2	...
3	8	15' 0"	2' 0"	...	3.4		6' 0"	1 $\frac{1}{4}$ "	7.5	...
3	9	22' 0"	9' 0"	...	4.4		10' 0"	1.2"	10.7	...
3-I	10	20' 0"	10' 0"	...	6.5		10' 0"	1 $\frac{1}{2}$ "	9.2	...
I	11	20' 0"	3' 4 $\frac{1}{2}$ "	...	0.0		5' 0"	1.1"	5.4	...
I	12	24' 6"	8' 9 $\frac{1}{2}$ "	...	2.1		10' 0"	1.2"	10.5	...
I	13	20' 9"	10' 3 $\frac{1}{2}$ "	...	5.7		10' 0"	1.25"	10.2	...
I	14	20' 7"	10' 1"	...	6.3	-2.5	5' 0"	1 $\frac{3}{8}$ "	8.5	...
I	15	20' 0"	7' 7"	...	6.2	...	8' 0"	1"	14.7	...
I	16	18' 6"	6' 10"	...	6.0		5' 0"	1"	9.2	...

TABLE 2. — *Continued.*TEST PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.
Oak Piles.

MOVE.	CHARACTER.		Formula Values at Other Embedments.	Reported Condition when Driven.	Actual Condition when Pulled.	Remarks.
	Sand.	Clay.				
.....			7.6 tons @ 3' 0"		Pulled OK	Easy driving
.....			9.1 tons @ 2' 0"			Easy driving
.....			7.9 tons @ 2' 0"			Easy driving
.....			7.0 tons @ 2' 6"			Easy driving
.....			6.2 tons @ 2' 0"			Easy driving
.....			9.2 tons @ 8' 6"			
.....						Easy driving
.....			11.0 tons @ 5' 0"			
.....						Easy driving
.....			11.5 tons @ 2' 9"			
.....			10.2 tons @ 3' 0"			
.....			12.7 tons @ 3' 0"		Point prob- ably in clay	Easy driving
.....			11.5 tons @ 2' 0"			
.....			12.3 tons @ 4' 0"			
.....			10.5 tons @ 2' 0"			

TABLE 3.

TEST PILES LOADED IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.
GROUP C.

IDENTITY.		Length of Pile.	EMBEDMENT.		SURFACE GRADES.		Ave. Drop Last Blows of Hammer.	Ave. Pen. Last 3 Blows.	Formula Load. Tons.	Test Load at Settlement. Tons.
Bldg. No.	Pile No.		Sand.	Clay.	Sand.	Clay.				
8	14a 5	35' 10"	11' 6"	7' 3"	-1.5	-13.0	2600 @ 10'	3 $\frac{3}{8}$ "	5.50	5.5 tons*
8	2a 4	30' 0"	13' 2"	0' 6"	-1.5	-14.7	2150 @ 10'	2 $\frac{1}{2}$ "	6.14	7 tons
8	12d 6	42' 6"	11' 6"	15' 4"	-1.5	-13.0	2600 @ 10'	3"	6.50	9 tons*
8	2a 2a	35' 9"	12' 6"	6' 2"	-1.5	-14.5	2150 @ 10'	2"	7.16	10 tons
6	12g 12	45' 0"	1' 6"	18' 5"	...		2600 @ 10'	3 $\frac{3}{8}$ "	5.90	15.43 tons
4	11a 6	50' 0"	0	21' 3"	0	-13.2	2580 @ 10'	4 $\frac{3}{4}$ "	4.50	12.80 tons
15	10a 18	42' 6"	0	23' 0"	0	-6.6	2300 @ 10'	3"	5.80	12.88 tons
7	13g 13	14' 9"	5' 10"	0	+5.60	0	2300 @ 10'	2 $\frac{1}{8}$ "	7.40	8.0 tons*
7	11d 2	15' 6"	5' 7"	0	+5.60	0	2300 @ 10'	1 $\frac{5}{8}$ "	8.80	9.0 tons*
Near 15	12	30' 0"	12' 0"	0	-0.7	-12.6	2300 @ 9.0'	2"	6.9	3.75 tons
Near 13	17	36' 0"	15' 6"	1.56'	-1.5	-17.0	2300 @ 10.0'	2"	7.7	9.0 tons*
Near 11	21	44' 0"	14' 6"	12.60'	-1.0	-15.5	2300 @ 11.0'	1 $\frac{1}{2}$ "	10.1	Embe
Near 16	26	30' 0"	11' 10"	1.95'	+1.4	-10.4	2300 @ 10.5'	2"	8.05	11.5 tons*
Near 2	29	30' 0"	10' 0"	0.5'	+1.3	-8.6	2300 @ 6.5'	$\frac{5}{8}$ "	9.02	9.0 tons*
17	n8 41	46' 0"	5' 6"	20.75'	-3.50	-9.0	2550 @ 10.0'	2"	8.50	9.8 tons
50	50-1	29' 9"	1' 6"	0	-7.99	-12.49	2600 @ 10.0'	1 $\frac{1}{2}$ "	10.40	7.40
50	50-2	30' 9"	1' 6"	0	-6.75	-11.25	2600 @ 10.0'	3"	6.50	7.30

* Found by interpolation.

TABLE 3.—Continued.

PILES LOADED IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

	CHARACTER.		Formula Values at Other Embedments.	Reported Condition when Driven.	Actual Condition when Pulled	Remarks.
	Sand.	Clay.				
ns.	Coarse and medium	Medium			OK	Allowable value limited by settlement
ns	Sand and fine gravel	Medium				
ns	Coarse and medium	Medium				
tons	Coarse and medium	Medium				
ns		Medium	Redriven value			13.90
ns	o	Stiff	Redriven value			11.50
ns	o	Medium				
ns	Sand	o			Pulled OK	
ns	Gravel	o			Pulled OK	
ns	Sand	o			Pulled OK	
ns	Gravel	o			Pulled OK	
ns	Fine sand		10 tons@10.5'	Uniform driving	Pulled OK OK	Early settlements appear excessive
ns	Fine sand		8.6 tons @ 12.0'	Easy driving	OK	
and	settlement	readings	of load test doubtful.			
ns	Sand	Blue medium	9.2 tons @ 10.0'	4' 6" of clay and shells above		the sand.
ns	Coarse sand	Clay and gravel	11.2 tons@10.0'	Uniform driving	Tip slightly broomed around edges	Early settlement readings in question
ns	Fine and coarse	Medium blue	Uniform increase in tonnage in clay.			
ons	Medium	o			Not pulled	Driven butt down
ons	Medium	o			Not pulled	Driven tip down

TABLE 4.

TEST OF PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

Identity.	Character.	Total Length in Ground (Feet).	Length in Sand, Gravel or Clay (in Feet).	Tip Diameter (in Inches).	Weight of Hammer (Lbs.).	Fall of Hammer (in Feet).	Penetration, Last Blow (in Inches).	Formula Value.	Test Load (Pounds).	Settlement (Inches).
14a 5 8	Spruce	32.8	Sand 11.5 Clay 7.25	5	2 600	10	3 $\frac{3}{4}$	..	6 990 11 970 26 390 31 640 36 900 53 641 66 504 72 925 77 403	0 $\frac{1}{16}$ $\frac{1}{8}$ $\frac{1}{4}$ $\frac{1}{2}$ $\frac{3}{4}$ $\frac{1}{2}$ $\frac{1}{4}$ $\frac{1}{8}$
2a 4 8	Spruce	27.7	Sand 13.2 Clay 0.5	6	2 150	10	2 $\frac{1}{2}$	..	13 920 18 690 24 510 29 860	$\frac{1}{16}$ $\frac{1}{8}$ $\frac{1}{4}$ $\frac{1}{8}$
12d 6 8	Spruce	40.9	Sand 11.5 Clay 15.33	6	2 600	10	3	..	7 090 14 350 20 388 25 241 29 861 35 068 45 068 59 161 65 509 70 686 75 228 80 108 84 834 89 814	0 $\frac{1}{16}$ $\frac{1}{8}$ $\frac{1}{4}$ $\frac{1}{2}$ $\frac{3}{4}$ $\frac{1}{2}$ $\frac{1}{4}$ $\frac{1}{8}$ $\frac{1}{4}$ $\frac{1}{8}$ $\frac{1}{16}$ $\frac{1}{8}$ $\frac{1}{16}$
2a 2a 8	Spruce	32.8	Sand 12.5 Clay 6.2	5 $\frac{1}{2}$	2 150	10	2	..	8 990 13 760 20 030 28 210 34 200 40 040 45 898 51 000 58 070 62 361	0 $\frac{3}{4}$ $\frac{1}{16}$ $\frac{1}{8}$ $\frac{1}{4}$ $\frac{1}{2}$ $\frac{3}{4}$ $\frac{1}{2}$ $\frac{1}{4}$ $\frac{1}{8}$ $\frac{1}{16}$

TABLE 5.

TEST OF PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

Identity.	Character.	Total Length in Ground (Feet).	Length in Sand, Gravel or Clay (in Feet).	Tip Diameter (in Inches).	Weight of Hammer (Lbs.).	Fall of Hammer (in Feet).	Penetration Last Blow (in Inches).	Formula Value.	Test Load (Pounds).	Settlement (Inches).
12g 12 6	Spruce	43.1	Sand 1.5 Clay 18.5	5	2 600	10	3.3	..	7 889 16 629 21 515 25 993 30 866 39 439 44 919 48 521 53 771 59 161 63 951 68 677 73 597 78 547 83 547	0 1/4 3/8 1/2 5/8 3/4 7/8 1 1 1/8 1 1/4 1 1/2 1 5/8 1 3/4 1 7/8 2 2 1/8 2 1/4 2 1/2 2 3/4 3 3 1/4 3 1/2 3 3/4 4 4 1/4 4 1/2 4 3/4 5 5 1/4 5 1/2 5 3/4 6 6 1/4 6 1/2 6 3/4 7 7 1/4 7 1/2 7 3/4 8 8 1/4 8 1/2 8 3/4 9 9 1/4 9 1/2 9 3/4 10 10 1/4 10 1/2 10 3/4 11 11 1/4 11 1/2 11 3/4 12 12 1/4 12 1/2 12 3/4 13 13 1/4 13 1/2 13 3/4 14 14 1/4 14 1/2 14 3/4 15 15 1/4 15 1/2 15 3/4 16 16 1/4 16 1/2 16 3/4 17 17 1/4 17 1/2 17 3/4 18 18 1/4 18 1/2 18 3/4 19 19 1/4 19 1/2 19 3/4 20 20 1/4 20 1/2 20 3/4 21 21 1/4 21 1/2 21 3/4 22 22 1/4 22 1/2 22 3/4 23 23 1/4 23 1/2 23 3/4 24 24 1/4 24 1/2 24 3/4 25 25 1/4 25 1/2 25 3/4 26 26 1/4 26 1/2 26 3/4 27 27 1/4 27 1/2 27 3/4 28 28 1/4 28 1/2 28 3/4 29 29 1/4 29 1/2 29 3/4 30 30 1/4 30 1/2 30 3/4 31 31 1/4 31 1/2 31 3/4 32 32 1/4 32 1/2 32 3/4 33 33 1/4 33 1/2 33 3/4 34 34 1/4 34 1/2 34 3/4 35 35 1/4 35 1/2 35 3/4 36 36 1/4 36 1/2 36 3/4 37 37 1/4 37 1/2 37 3/4 38 38 1/4 38 1/2 38 3/4 39 39 1/4 39 1/2 39 3/4 40 40 1/4 40 1/2 40 3/4 41 41 1/4 41 1/2 41 3/4 42 42 1/4 42 1/2 42 3/4 43 43 1/4 43 1/2 43 3/4 44 44 1/4 44 1/2 44 3/4 45 45 1/4 45 1/2 45 3/4 46 46 1/4 46 1/2 46 3/4 47 47 1/4 47 1/2 47 3/4 48 48 1/4 48 1/2 48 3/4 49 49 1/4 49 1/2 49 3/4 50 50 1/4 50 1/2 50 3/4 51 51 1/4 51 1/2 51 3/4 52 52 1/4 52 1/2 52 3/4 53 53 1/4 53 1/2 53 3/4 54 54 1/4 54 1/2 54 3/4 55 55 1/4 55 1/2 55 3/4 56 56 1/4 56 1/2 56 3/4 57 57 1/4 57 1/2 57 3/4 58 58 1/4 58 1/2 58 3/4 59 59 1/4 59 1/2 59 3/4 60 60 1/4 60 1/2 60 3/4 61 61 1/4 61 1/2 61 3/4 62 62 1/4 62 1/2 62 3/4 63 63 1/4 63 1/2 63 3/4 64 64 1/4 64 1/2 64 3/4 65 65 1/4 65 1/2 65 3/4 66 66 1/4 66 1/2 66 3/4 67 67 1/4 67 1/2 67 3/4 68 68 1/4 68 1/2 68 3/4 69 69 1/4 69 1/2 69 3/4 70 70 1/4 70 1/2 70 3/4 71 71 1/4 71 1/2 71 3/4 72 72 1/4 72 1/2 72 3/4 73 73 1/4 73 1/2 73 3/4 74 74 1/4 74 1/2 74 3/4 75 75 1/4 75 1/2 75 3/4 76 76 1/4 76 1/2 76 3/4 77 77 1/4 77 1/2 77 3/4 78 78 1/4 78 1/2 78 3/4 79 79 1/4 79 1/2 79 3/4 80 80 1/4 80 1/2 80 3/4 81 81 1/4 81 1/2 81 3/4 82 82 1/4 82 1/2 82 3/4 83 83 1/4 83 1/2 83 3/4 84 84 1/4 84 1/2 84 3/4 85 85 1/4 85 1/2 85 3/4 86 86 1/4 86 1/2 86 3/4 87 87 1/4 87 1/2 87 3/4 88 88 1/4 88 1/2 88 3/4 89 89 1/4 89 1/2 89 3/4 90 90 1/4 90 1/2 90 3/4 91 91 1/4 91 1/2 91 3/4 92 92 1/4 92 1/2 92 3/4 93 93 1/4 93 1/2 93 3/4 94 94 1/4 94 1/2 94 3/4 95 95 1/4 95 1/2 95 3/4 96 96 1/4 96 1/2 96 3/4 97 97 1/4 97 1/2 97 3/4 98 98 1/4 98 1/2 98 3/4 99 99 1/4 99 1/2 99 3/4 100 100 1/4 100 1/2 100 3/4 101 101 1/4 101 1/2 101 3/4 102 102 1/4 102 1/2 102 3/4 103 103 1/4 103 1/2 103 3/4 104 104 1/4 104 1/2 104 3/4 105 105 1/4 105 1/2 105 3/4 106 106 1/4 106 1/2 106 3/4 107 107 1/4 107 1/2 107 3/4 108 108 1/4 108 1/2 108 3/4 109 109 1/4 109 1/2 109 3/4 110 110 1/4 110 1/2 110 3/4 111 111 1/4 111 1/2 111 3/4 112 112 1/4 112 1/2 112 3/4 113 113 1/4 113 1/2 113 3/4 114 114 1/4 114 1/2 114 3/4 115 115 1/4 115 1/2 115 3/4 116 116 1/4 116 1/2 116 3/4 117 117 1/4 117 1/2 117 3/4 118 118 1/4 118 1/2 118 3/4 119 119 1/4 119 1/2 119 3/4 120 120 1/4 120 1/2 120 3/4 121 121 1/4 121 1/2 121 3/4 122 122 1/4 122 1/2 122 3/4 123 123 1/4 123 1/2 123 3/4 124 124 1/4 124 1/2 124 3/4 125 125 1/4 125 1/2 125 3/4 126 126 1/4 126 1/2 126 3/4 127 127 1/4 127 1/2 127 3/4 128 128 1/4 128 1/2 128 3/4 129 129 1/4 129 1/2 129 3/4 130 130 1/4 130 1/2 130 3/4 131 131 1/4 131 1/2 131 3/4 132 132 1/4 132 1/2 132 3/4 133 133 1/4 133 1/2 133 3/4 134 134 1/4 134 1/2 134 3/4 135 135 1/4 135 1/2 135 3/4 136 136 1/4 136 1/2 136 3/4 137 137 1/4 137 1/2 137 3/4 138 138 1/4 138 1/2 138 3/4 139 139 1/4 139 1/2 139 3/4 140 140 1/4 140 1/2 140 3/4 141 141 1/4 141 1/2 141 3/4 142 142 1/4 142 1/2 142 3/4 143 143 1/4 143 1/2 143 3/4 144 144 1/4 144 1/2 144 3/4 145 145 1/4 145 1/2 145 3/4 146 146 1/4 146 1/2 146 3/4 147 147 1/4 147 1/2 147 3/4 148 148 1/4 148 1/2 148 3/4 149 149 1/4 149 1/2 149 3/4 150 150 1/4 150 1/2 150 3/4 151 151 1/4 151 1/2 151 3/4 152 152 1/4 152 1/2 152 3/4 153 153 1/4 153 1/2 153 3/4 154 154 1/4 154 1/2 154 3/4 155 155 1/4 155 1/2 155 3/4 156 156 1/4 156 1/2 156 3/4 157 157 1/4 157 1/2 157 3/4 158 158 1/4 158 1/2 158 3/4 159 159 1/4 159 1/2 159 3/4 160 160 1/4 160 1/2 160 3/4 161 161 1/4 161 1/2 161 3/4 162 162 1/4 162 1/2 162 3/4 163 163 1/4 163 1/2 163 3/4 164 164 1/4 164 1/2 164 3/4 165 165 1/4 165 1/2 165 3/4 166 166 1/4 166 1/2 166 3/4 167 167 1/4 167 1/2 167 3/4 168 168 1/4 168 1/2 168 3/4 169 169 1/4 169 1/2 169 3/4 170 170 1/4 170 1/2 170 3/4 171 171 1/4 171 1/2 171 3/4 172 172 1/4 172 1/2 172 3/4 173 173 1/4 173 1/2 173 3/4 174 174 1/4 174 1/2 174 3/4 175 175 1/4 175 1/2 175 3/4 176 176 1/4 176 1/2 176 3/4 177 177 1/4 177 1/2 177 3/4 178 178 1/4 178 1/2 178 3/4 179 179 1/4 179 1/2 179 3/4 180 180 1/4 180 1/2 180 3/4 181 181 1/4 181 1/2 181 3/4 182 182 1/4 182 1/2 182 3/4 183 183 1/4 183 1/2 183 3/4 184 184 1/4 184 1/2 184 3/4 185 185 1/4 185 1/2 185 3/4 186 186 1/4 186 1/2 186 3/4 187 187 1/4 187 1/2 187 3/4 188 188 1/4 188 1/2 188 3/4 189 189 1/4 189 1/2 189 3/4 190 190 1/4 190 1/2 190 3/4 191 191 1/4 191 1/2 191 3/4 192 192 1/4 192 1/2 192 3/4 193 193 1/4 193 1/2 193 3/4 194 194 1/4 194 1/2 194 3/4 195 195 1/4 195 1/2 195 3/4 196 196 1/4 196 1/2 196 3/4 197 197 1/4 197 1/2 197 3/4 198 198 1/4 198 1/2 198 3/4 199 199 1/4 199 1/2 199 3/4 200 200 1/4 200 1/2 200 3/4 201 201 1/4 201 1/2 201 3/4 202 202 1/4 202 1/2 202 3/4 203 203 1/4 203 1/2 203 3/4 204 204 1/4 204 1/2 204 3/4 205 205 1/4 205 1/2 205 3/4 206 206 1/4 206 1/2 206 3/4 207 207 1/4 207 1/2 207 3/4 208 208 1/4 208 1/2 208 3/4 209 209 1/4 209 1/2 209 3/4 210 210 1/4 210 1/2 210 3/4 211 211 1/4 211 1/2 211 3/4 212 212 1/4 212 1/2 212 3/4 213 213 1/4 213 1/2 213 3/4 214 214 1/4 214 1/2 214 3/4 215 215 1/4 215 1/2 215 3/4 216 216 1/4 216 1/2 216 3/4 217 217 1/4 217 1/2 217 3/4 218 218 1/4 218 1/2 218 3/4 219 219 1/4 219 1/2 219 3/4 220 220 1/4 220 1/2 220 3/4 221 221 1/4 221 1/2 221 3/4 222 222 1/4 222 1/2 222 3/4 223 223 1/4 223 1/2 223 3/4 224 224 1/4 224 1/2 224 3/4 225 225 1/4 225 1/2 225 3/4 226 226 1/4 226 1/2 226 3/4 227 227 1/4 227 1/2 227 3/4 228 228 1/4 228 1/2 228 3/4 229 229 1/4 229 1/2 229 3/4 230 230 1/4 230 1/2 230 3/4 231 231 1/4 231 1/2 231 3/4 232 232 1/4 232 1/2 232 3/4 233 233 1/4 233 1/2 233 3/4 234 234 1/4 234 1/2 234 3/4 235 235 1/4 235 1/2 235 3/4 236 236 1/4 236 1/2 236 3/4 237 237 1/4 237 1/2 237 3/4 238 238 1/4 238 1/2 238 3/4 239 239 1/4 239 1/2 239 3/4 240 240 1/4 240 1/2 240 3/4 241 241 1/4 241 1/2 241 3/4 242 242 1/4 242 1/2 242 3/4 243 243 1/4 243 1/2 243 3/4 244 244 1/4 244 1/2 244 3/4 245 245 1/4 245 1/2 245 3/4 246 246 1/4 246 1/2 246 3/4 247 247 1/4 247 1/2 247 3/4 248 248 1/4 248 1/2 248 3/4 249 249 1/4 249 1/2 249 3/4 250 250 1/4 250 1/2 250 3/4 251 251 1/4 251 1/2 251 3/4 252 252 1/4 252 1/2 252 3/4 253 253 1/4 253 1/2 253 3/4 254 254 1/4 254 1/2 254 3/4 255 255 1/4 255 1/2 255 3/4 256 256 1/4 256 1/2 256 3/4 257 257 1/4 257 1/2 257 3/4 258 258 1/4 258 1/2 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TEST OF PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

[illegible]

TABLE 7.

TEST OF PILES IN NEW SITE OF MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

Identity.	Character.	Total Length in Ground (Feet).	Length in Sand, Gravel or Clay (in Feet).	Tip Diameter (in Inches).	Weight of Hammer (Lbs.).	Fall of Hammer (in Feet).	Penetration Last Blow (in Inches).	Formula Value.	Test Load (Pounds).	Settlement (Inches).
No. 1	Raymond Concrete	23.2	Sand and gravel 10.2	8	No. 1 Vulcan. Weight of moving parts 5 000 lbs.	2.25	5 blows per inch.	..	41 050	0
.....	Near No. 14 Wood Pile given by Mr. Worcester.								46 070	$\frac{1}{16}$
.....									61 810	$\frac{1}{16}$
.....									81 840	$\frac{5}{8}$
.....									86 710	$\frac{3}{2}$
.....									90 890	$\frac{5}{8}$
.....									100 330	$\frac{3}{2}$
.....									125 340	$\frac{1}{8}$
.....									144 770	$\frac{1}{4}$
.....									171 750	$\frac{1}{32}$
.....									182 120	$\frac{1}{32}$
No. 2	Raymond Concrete	17.5	Sand and gravel 6.4	8	No. 1 Vulcan	2.25	blows per inch.		29 650	0
.....									44 420	$\frac{1}{16}$
.....									59 360	$\frac{3}{8}$
.....									110 630	$\frac{1}{16}$
.....									122 280	$\frac{1}{4}$
.....									126 700	$\frac{1}{16}$
.....									131 610	$\frac{3}{8}$
.....									158 104	$\frac{1}{32}$
.....									169 980	$\frac{1}{16}$
.....									175 870	$\frac{1}{32}$
No. 4	Simplex Concrete	12.4	Sand and gravel 4.0	16	3 300	12	$\frac{1}{8}$		42 490	$\frac{1}{8}$
.....									61 280	$\frac{1}{8}$
.....									85 800	$\frac{1}{16}$
.....									95 910	$\frac{3}{16}$
.....									133 790	$\frac{1}{16}$
.....									153 810	$\frac{1}{8}$
.....									177 020	$\frac{9}{32}$
.....									187 270	$\frac{1}{32}$
.....									192 270	$\frac{7}{16}$

carried at $\frac{1}{4}$ -in. settlement was divided by $2\frac{1}{2}$, and the result used for the new value of the pile.

Piles enclosed in squares on Fig. 8 were load tested, some of the most important of which are recorded in full in Tables 4, 5, 6 and 7.

After determining that wood piles were to be used, it became necessary to make a sufficient number of tests in order to determine the kind and length to use in different parts of the site.

This task was rather difficult, owing to the great variety of soil conditions found, as will be seen by reference to the geological sections and pile driving reports.

It will be noted that the investigations were first carried on largely on the sites of the buildings forming the westerly part of the group where both spruce and oak test piles were driven.

Referring to Table 1, which is a summary of spruce test piles driven and tested, it will be seen that spruce piles Nos. 1 to 10 inclusive were driven into the "glacial gravel," which at these locations is a well compacted and deep bed of material commonly known as coarse sand and gravel, — to varying depths, and upon being pulled were all found to be broomed or both broomed and broken. Three of these piles, Nos. 3, 5 and 7, were assumed to be good at the time of driving, and were load tested before being pulled.

The results of such tests, because of their crippled condition, which was discovered after the tests were made, were of no special value except to show how these broomed and broken piles acted under load, and the following brief description of these is given for that purpose.

Spruce pile No. 3, about 17 ft. 0 in. long, 9 in. diameter butt and 6 in. tip, driven with 2300-lb. hammer and given short drops until the last few blows, where an average of 9 ft. 8 in. drop was given and a value of 16.7 tons obtained when using the *Engineering News* formula. This pile passed through about 10 ft. 6 in. of loose sand, silt, shells and blue-black mud, and penetrated into coarse sand and gravel about 3 ft. 9 ins.; but when pulled, after load testing, was found broken at about the surface of the gravel and badly broomed at the break.

When tested with a load, this pile, No. 3, carried 5.5 tons

without noticeable settlement; but when this load was doubled, it settled $\frac{1}{4}$ in. As would be expected, the settlements were marked, and under a total load of 48 674 ran from $\frac{1}{3}\frac{1}{2}$ in. to 1 in.

Pile No. 5. Spruce, having an 11 in. butt, 6 in. tip, driven with 2300-lb. hammer and with an average drop of hammer of 6 ft. 2 ins. for the last blows, showed an average settlement of $\frac{5}{8}$ in. per blow, which gives a formula value of 8.77 tons.

This pile passed through about 9 ft. 7 ins. of loose sand, silt, shells, silt and black mud, and penetrated the coarse sand and gravel for about 5 ft. 1 in., but when pulled was found badly broomed and broken just below the top of the gravel.

When tested with a load, this pile carried 6.8 tons with a total settlement of only $1/32$ in., but it settled quite rapidly for increasing loads.

Under about 18 tons load, the settlement was $\frac{7}{16}$ in. and showed a marked tendency to get out of plumb. Under about 20 tons the total settlement amounted to over 1 in.

This pile showed great unreliability.

Pile No. 7. Spruce, about 18 ft. 4 ins. long, 10 in. butt by 6 in. tip, driven with 2300-lb. hammer having an average drop at last blows of 10 ft. 0 in., and an average settlement of $1\frac{3}{4}$ in. at last blows, which gives a carrying value of 8.4 tons.

This pile passed through about 9 ft. 2 ins. of loose sand, gravel, shells and mud fill, and penetrated the glacial gravel deposit about 5 ft. 9 ins., and when pulled was found broken and broomed just below top of the gravel.

When tested with a load, this pile settled $\frac{1}{16}$ in. under a load of about three tons, and the rate of settlement was quite rapid under succeeding increments, amounting to over 1 in. under about 15 tons, to over 2 ins. under 26.5 tons, and over 3 ins. under 32.75 tons.

After the load was removed, this pile recovered $\frac{1}{2}$ in. of settlement, showing the effect of the brooming.

It was plainly evident that spruce piles could not be driven, with safety, into the harder portions of this glacial deposit. After a study of the results obtained by a number of spruce piles which had been driven and pulled, it was thought best to limit the use of spruce to those places where friction was largely

depended upon to give the bearing value and very little dependence placed on point bearing.

The test piles also showed that it was easily possible to drive spruce piles, which appeared to be good to the piling foreman and inspector, but which were actually found to be broomed and broken when pulled.

This discouraged the use of spruce piles where sudden changes in the hardness of the soil were found or expected.

Some of the marked and unaccountable settlements of structures which have been recorded in the past may have been due to some extent to the practice of driving spruce piles to or somewhat into hard crusts, thus resulting in brooming of the piles.

Later tests also proved that first-class oak piles could be driven without injury into well-compacted coarse sand or fine gravel to a resistance giving a value of eighteen tons or more.

Oak piles were used where the driving was hard, and where the supporting value of the piles was to be gained largely from point bearing and relatively small embedments in the hard stratum.

For safety a maximum limit of about ten tons was permitted for spruce and fourteen tons for oak.

The *Engineering News* formula was used in determining values of piles for these foundations, and for convenience curves were made up showing safe loads for piles for various weights and drops of hammers.

This formula is as follows:

$$P = \frac{2Wh}{S+1}, \text{ in which}$$

P = Supporting power in pounds.

W = Weight of hammer in pounds.

h = Fall of hammer in feet.

S = Penetration in inches.

Referring to Figs. 2, 3, 4, 5 and 6, it will be seen that a great variety of soil conditions was found.

The glacial deposit ranged from a well compacted gravel and coarse sand, to fine sand, and from 35 ft. in thickness to

nothing, so that in parts of some buildings it was necessary to depend upon the clay for support.

A hard crust was found in only a few locations on either the "glacial gravel" or clay.

The clay in general was found to be of a remarkably uniform grade, designated as "medium," being neither very "stiff" or "soft" except in a few places.

From the driving records and sections it would seem that the surface of the glacial deposit was very uneven in places, as piles which were driven within a few feet of each other would find good bottom at levels differing by 10 to 15 ft. or even more.

These conditions made it difficult to get uniform pile values, and increased the cost of the pile driving.

A fill was often found over the original mud flats, consisting of material such as mud, silt, shells and gravel, which in some places was very compact, offering considerable resistance to the piles, and raising the formula value of all such piles above the true value.

This high but untrue value was corrected to some extent by subtracting the value found while the pile is still in the fill, from the final value.

Not only will the hard fill, above the peat, etc., give the piles driven through it a high formula value at the time of driving, but may afterward add to the actual loads on the piles, by reason of its friction when the peat or mud stratum becomes reduced in thickness, through decay or displacement.

The varying conditions found made it necessary to take particular care to design the pile foundations on the basis of as nearly uniform settlements as possible. This uniformity is highly desirable in order to prevent over-stressing the more or less continuous concrete floor beams and slabs used in the superstructure of the buildings.

The working values have been taken at about $\frac{1}{16}$ -in. settlement, as shown by the tests, as it is believed that most piles have an initial settlement, whether noted or not, and considerable care was taken at these tests to note settlements at all times of changes as small as $\frac{1}{16}$ in. It is also believed that the effect of a difference in settlement of $\frac{1}{16}$ in. in the foundations can be safely ignored.

A settlement of $\frac{1}{4}$ in. was considered to be the limit of usefulness of a pile, and it was assumed that greater settlements than $\frac{1}{4}$ in. might create conditions which would cause very unsatisfactory results.

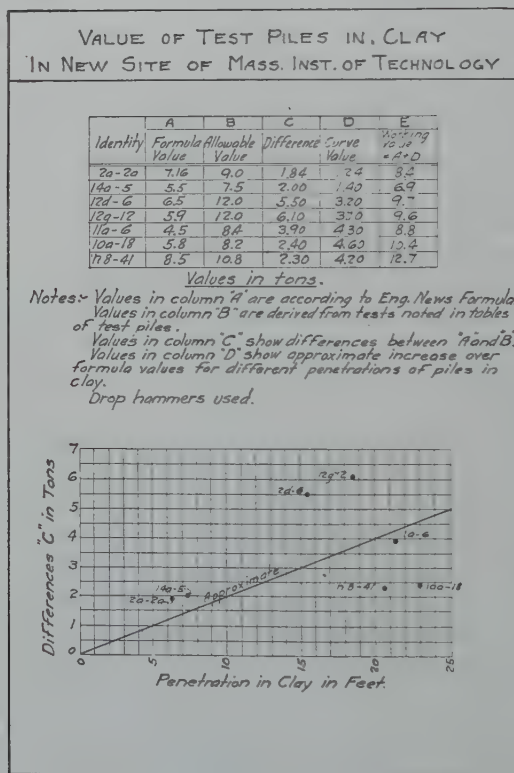


FIG. 9.

It was also considered necessary that the piling have a safety factor of not less than 2.5 of tonnage, based on the limit of $\frac{1}{4}$ -in. settlement. From the results of the test piles it is evident that in the majority of cases the piles had to be driven quite deeply into the glacial deposit in order to assure a satisfactory tonnage, but the effort was made to keep the piles in the glacial deposit, even where it thinned up, as it was desirable to use this

stratum as a medium for spreading the loads over the clay under it. An effort was made to have a minimum of at least three feet of "gravel" under the points even where it became thin, in order to make the spreading of the load somewhat effective. It was found, however, that there were places where the gravel was very thin, giving very little value to the piles, and if depended on alone would make the piers over them large and expensive.

When the glacial deposit was found to be thin or disappeared altogether, it then became necessary to depend directly on the clay for support.

The clay, with almost no exception, was without a hard surface or crust, and in order to ascertain its value under working and ultimate loads, some piles, which were driven both partly and wholly into the clay, were load tested.

Two of these were also redriven, as well as a number of others not shown.

Regarding the safe working values, it was found, from the tests, that piles driven into sand and depending upon friction for support, could be used at their formula value, but not much higher, for the limits of settlements and safety factor assumed.

On the other hand, piles embedded in the clay found at this site showed an ability to carry test loads in excess of the formula value when using drop hammers. Referring to the curve shown on Fig. 18, it will be seen that the allowable increase of the formula value appears to be about one ton for every five feet of embedment in the clay. These tests would appear to allow us to believe that, for drop hammers driving wood piles into clay such as found at this site, the *Engineering News* formula could be modified and used as $\frac{3Wh}{S+1}$.

This was taken full advantage of where the gravel and sand became either unreliable, of no practical value, or disappeared, as in a part of Building 17 and others.

Piers were frequently made in clay in which the average formula value of its piles was six tons and less, but owing to the high value shown by tests these were allowed a value fully 50 per cent. greater, provided it was evident that they were well embedded in the clay.

An examination of the reports of piles 12, 6 and 18, in Buildings 6, 4 and 15, respectively, shows a high tonnage at $\frac{1}{16}$ -in. settlement; but when compared with the tonnage at $\frac{1}{4}$ -in. settlement, a low factor of safety is found; but when reduced to get a factor of about 2.5, it will be seen that there is still a very substantial amount above the formula tonnage, found at time of driving.

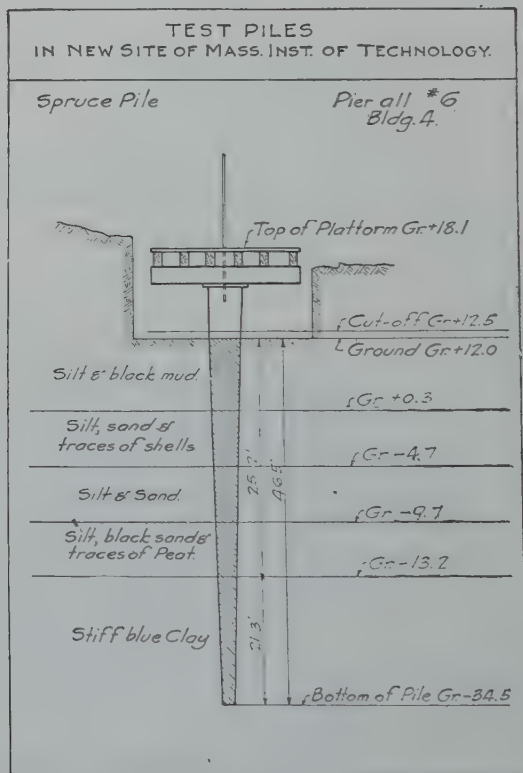


FIG. 10.

It is interesting to see, by the table, that the formula tonnage found by re-driving the piles, after they had been allowed to thoroughly set, did not represent either the value under test load; at $\frac{1}{16}$ -in. settlement, or the "allowable" tonnage.

Figs. 10, 11 and 12 illustrate some of the wood test piles and the character of the soil in which they are embedded.

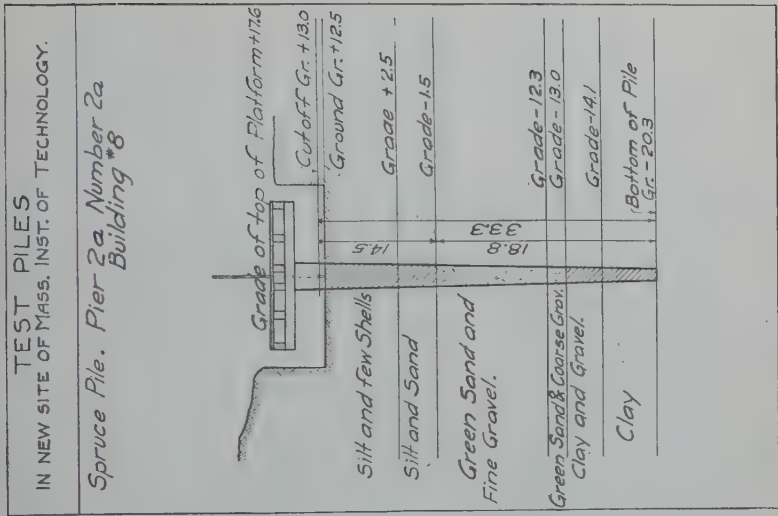


FIG. 12.

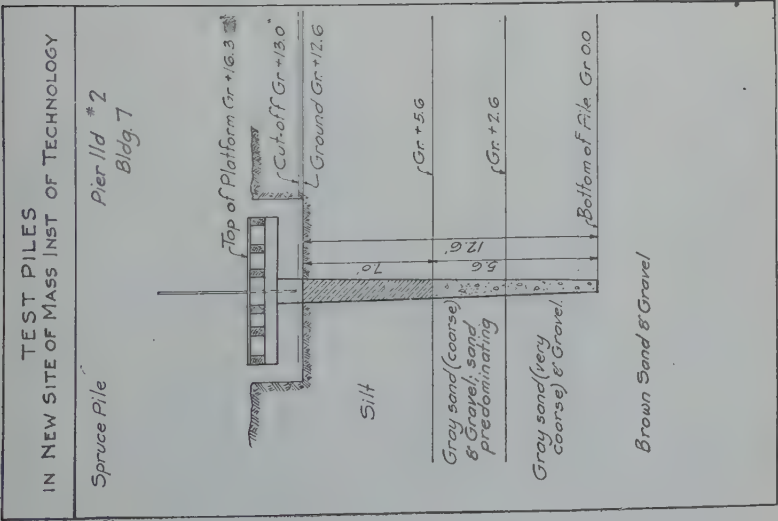


FIG. 11.

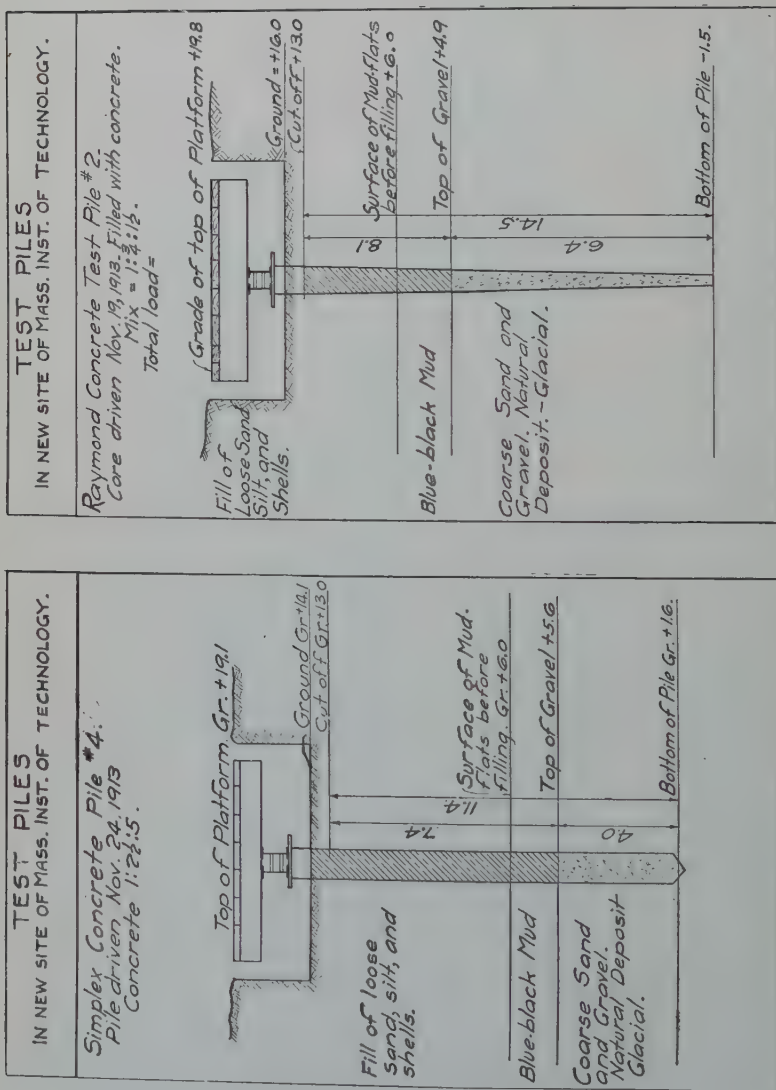


FIG. 13.

FIG. 14.

Fig. 13 shows one Simplex pile which was load tested, and the soil in which it rested.

Fig. 14 shows one Raymond pile under similar conditions.

Another feature, in the problem of these foundations, which was given some consideration was the matter of vibration, due to internal and external forces.

Without question it was necessary to construct the foundations in such a way as to reduce the possibility of vibration to a minimum, not only for comfort but because some of the laboratories will contain instruments of special accuracy and delicacy.

Special care has been taken regarding the foundations of heavy testing machinery, to reduce the possibility of settlement and excessive vibration, by placing a sufficient number of well-driven piles under such machines and allowing about 50 per cent. of the value given piles under the building walls.

The greatest vibration will come to the structures from external sources, such as engines in nearby power houses, freight trains, electric cars, etc., conveyed to the buildings by means of the comparatively hard top fill which lies over the stratum of mud, silt and peat.

Peat is found in some locations within the building site, and is probably general on the north and east sides of the buildings, extending through that portion of East Cambridge where a number of factories and power houses are located. Nearly the entire site of the new buildings has a stratum of silt, mud and fill above the glacial deposit and clay. The buildings have therefore a foundation of piles which for their entire lengths, above that part embedded in sand and gravel or clay, pass through a material affording very little lateral support.

Under the action of driving piles, this material often softened up to such an extent as to allow the piles to vibrate strongly, often throwing mud and water out around them.

Structures which are supported on piles passing through such soft, water-filled strata, not only readily send out vibrations but are strongly affected by vibrations from other sources, if situated along the same strata.

One method of partially resisting such a tendency is to

drive the piles well into the supporting stratum, rather than resting them on the surface or with a foot or two of penetration.

As the pile tests show that it was generally necessary to get a fair embedment, in order to get the bearing values desired, it was made unnecessary to largely increase that embedment in order to gain stiffness.

In general, the unit stresses per pile averaged 9.5 tons for interior piers and one-half to three-quarters of a ton less for exterior piers and walls.

It is believed that if settlements occur, the small difference in loading will tend to make slightly greater settlements in the center of the building and prevent the bulging tendency of walls, which would occur if the wall piles settled more than the interior piles.

The working rules for governing the driving of piles are as follows:

1. Spruce piles should not be driven when "hard" driving is found, irrespective of the cause.

2. Driving shall be called "hard" when the penetration per blow of or equivalent of a 2300-lb. hammer at a drop of 10 ft. is less than $1\frac{1}{4}$ in. (which equals 10.2 tons by the *Engineering News* formula).

3. Where greater penetrations are found with equivalent weight of hammer and drop, it shall then be called "easy" driving.

4. When "easy" driving is found, all piles should be driven into the glacial drift 12 ft., provided an average bearing value of 10 tons is obtained in the group.

5. If a 10-ton average cannot be obtained, then longer piles shall be driven to greater depths *in the glacial drift*, in order to get an average of 10 tons. The minimum value for averages should be 8 tons; the maximum, 11 tons, for spruce.

6. If an average value of 10 tons cannot be obtained in any group or pier with a reasonably long pile, say up to 50 ft., then it will be best to stop the driving at such a place, and get a new design for the group, based on values which are indicated.

7. Use oak piles for all "hard" driving and at other places as hereinafter specified.

8. Drive oak piles into the "drift" at least 7 ft., if possible, but do not exceed a value of 14 tons, in attempting to get that penetration.

9. If oak piles do not show a value of 10 tons when driven into "drift" 7 ft., then drive deeper until it is obtained.

10. Do not drive piles through the drift into the clay, as the bearing power must be obtained in the stratum of glacial drift if possible.

11. In those parts of the job where there is any possibility of going through the drift, proceed as follows:

Drive a long, oak test pile in the center of each group of piles supporting each column, with the object of finding the probable depth of the drift.

If the drift is not thick enough to allow spruce or oak piles to be driven in the regular way, then deduct 3 ft. from the thickness of the drift and drive all piles to the new grade, thus keeping all pile points 3 ft. above the clay.

If this results in a low average value for the group, then more piles must be driven to bring the total value of the group up to the required total.

The question whether spruce or oak shall be used in such a case will be determined by the same rules as for any other parts of the work.

If at any time the drift tightens up, so that the penetration per blow is below $1\frac{1}{4}$ ins. when using a drop hammer weighing 2300 lb. at a drop of 10 ft. 0 in. or the equivalent, then use oak.

12. Where a hard fill is found above the glacial drift, it will be necessary to drive oak piles into the glacial drift the usual distance, regardless of the fill.

13. Where the drift is thin or not found, and clay depended upon, piles are to be driven having not less than 18 ft. embedment for medium clay or equivalent, and not more than 9.5 tons shall be allowed them in designing the footings.

FOUNDATIONS ABOVE PILES.

Reinforced concrete spread footings were used to distribute the wall and column loads to the piles. Such footings are per-

missible only when placed on a compressible surface of practically uniform resistance.

This will produce bending stresses in the footings, similar to those produced by a uniformly distributed load.

The permanent ground water level was such as to permit

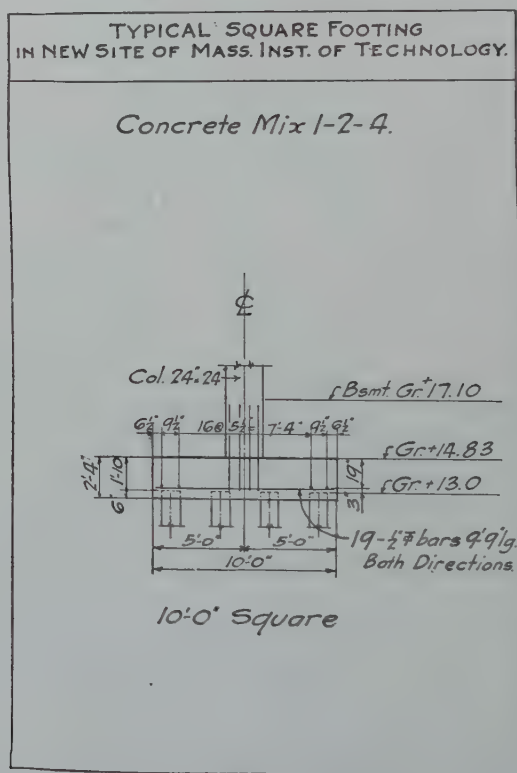


FIG. 15.

the cut-off grade of piles to be fixed at +13.00 Cambridge datum, and is 4.10 ft. below the basement floor level. The permanent water level is not less than +13.00 Cambridge datum.

With few exceptions, the difference of 4.10 ft. between floor and pile cut-off permitted the reinforced spread footings to be placed without lowering the cut-off.

The design of these footings was based on the formulæ recommended by Prof. Arthur N. Talbot, of the University of Illinois, in Bulletin No. 67, the formulæ being determined from the result of many experiments.

The working stresses are those recommended in the report

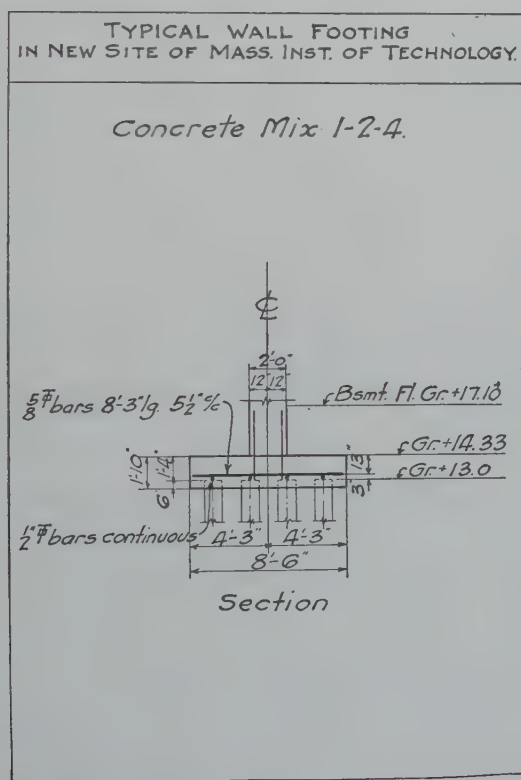


FIG. 16.

of the Joint Committee on Concrete and Reinforced Concrete, except that a maximum of 18 000 lbs. per sq. in. tension was allowed for square rods, cold twisted.

With this general type of footing it is recognized that the piles immediately under the load will be stressed somewhat higher than those at the edges, but this tendency was reduced

to some extent by making the depth of the large footings somewhat greater than strictly required.

This is especially necessary in cases of oblong footings, where the dimensions are 20 ft. by 10 ft., for example, in order that the piles directly under the column shall not be excessively loaded.

The reinforcing steel used in these foundations was mostly square bars, cold twisted.

The pile heads were covered with concrete to a distance of 6 in. below their tops, while the steel was placed with at least 2 in. of clearance above the tops of piles, affording ample protection against rusting or deterioration of any kind.

Figs. 15 and 16 show typical square and wall footings.

DISCUSSION.

DESMOND FITZGERALD.*—This paper by Mr. Main is most interesting. It is remarkable for the care with which the character of the ground was studied and the effort to meet the conditions in each case. I particularly admire the plan of driving the piles well into the clay to ensure a fair bearing, rather than to stop driving when the surface of the clay was reached.

I am afraid that I cannot add much of value to the discussion. My own experience in the driving of piles in Boston is confined almost entirely to the years 1871 and 1872, when I was engineer of the Boston & Albany Railroad. Many of our walls and abutments, round houses and other buildings, were founded on piles. The heaviest structures that I built at that time were the grain elevators at East Boston and at Berkeley Street, not far from the Providence Railroad crossing. In the case of the latter elevator, we used spruce piles and drove through the marsh to a layer of blue clay about $2\frac{1}{2}$ ft. thick, under which was a softer deposit. The driving of the piles was stopped on the $2\frac{1}{2}$ -ft. layer, as the resistance to the hammer seemed to be good at that point. Although the weight of the elevator, when filled with grain, was very great, no settlement was ever noticed in the heavy brick walls nor in the piers under the bins. The

* Consulting Engineer, Brookline, Mass.

foundations of the tower for Trinity Church were put in soon after the period of which I speak, and it would be interesting to find the old plans and to see the weight on each pile. As I remember, the foundation was spread to cover a large area. I think the site was in the deepest mud hole in the Back Bay, and an old channel ran directly under the site of the tower.

MR. LEWIS M. HASTINGS.* I have been very much interested in Mr. Main's paper. I shall also be interested to know how the results of the observation as to the settlement of these buildings come out. It would be a very great achievement in designing if the settlement should be uniform over this large spread of buildings, requiring such a wide diversity of foundation and such a multiplicity of design, — to meet those conditions. The local conditions there are rather unusual and interesting for an engineer. For instance, the Metropolitan Storage Warehouse near the Boston & Albany Railroad is a very large building and very heavy, four or five stories high, and of heavy brick construction. Under the whole of it, they drove piles to what was supposed to be a solid foundation. The building has been constructed in three portions, — the first portion on Massachusetts Avenue was built more than twenty years ago; the middle portion was built some time after that; and again another addition was made toward the southwest still later. That building has had a progressive settlement ever since the first observation was taken on it. We had a heavy bolt set in the building and intended to use that as a bench mark, about twenty years ago. The building has settled something like 1.295 ft. in that time. It might be expected that the building would be a wreck, but it is perfectly intact, and there is hardly a crack in it. That has been a progressive settlement, — progressive in amount but decreasing in rate, — so it seems to be coming down to a rather hard bottom now. Compare that with the result on the pier or abutment wall of Harvard Bridge. That pier or abutment at the sea wall is founded on the gravel without any piles or attempt to carry it on anything but the gravel. There we have also a bench on the capstone of the abutment,

* City Engineer, City Hall, Cambridge, Mass.

and that has not moved in over twenty years. That is a long way out in the river from the Metropolitan Storage Warehouse. That is a very striking anomaly in conditions, — the inland foundation has gone down continuously, while the one further in the river is practically rigid. We have tested that bench from the shore benches repeatedly, and have never been able yet to detect any essential settlement. Perhaps the explanation may be that it is founded on a very deep gravel bed. The gravel there is unusually heavy and deep, making the overlay on this clay a very heavy one. Further inland the gravel overlay is thinner and is nearer the glacial drift and clay that is underneath, extending to bed rock. As you go further toward the northwest, over to the other side of the Boston & Albany Railroad, we have two quite remarkable structures as far as their present condition is concerned. One is the metropolitan sewer, which passes through Portland Street a distance of about five hundred feet from the Metropolitan Storage Warehouse; and the other is the Cambridge city sewer, which passes along parallel with the metropolitan sewer in Portland Street. The metropolitan sewer is quite deep, going down into the natural sand, — through the filling and peat overlay, and into the natural sand. The city sewer was not so deep, but the foundation was carried down to the same sand with masonry or piling. Now, at the point near the junction of Albany and Portland streets, the maximum settlement was about three and a quarter feet. At "Fort Washington," about three thousand feet southwesterly, the sewer remains practically at grade, and at Broadway near the Mason and Hamlin Organ Company Building, about sixteen hundred feet northeasterly, it is at grade again, — so that between these points the metropolitan sewer is working as an inverted siphon. The city sewer is in the same condition at this point, being now about three feet below the original grade. At the point of greatest settlement above referred to, a boring was made by Past President Charles R. Gow, and it was carried one hundred forty-three feet below the surface to bed rock, and five feet more into the rock. From the level of the metropolitan sewer down to the rock, nothing but sand and clay was found by Mr. Gow. An-

other curious thing is that this settlement extends in a westerly direction away from Charles River and its muddy shores, for more than half a mile, to where the soil, thoroughly tested by borings for the "Cambridge Main Street Subway," consists entirely of sand and the underlying clay. Even here substantial settlements have occurred. Heavy brick buildings founded on the sand have settled from one to two feet without rupture of walls or foundations. The sewer in Main Street, running near the northerly boundary of the Technology property, was found to be from one to two feet below the grade at which it was laid although it was built in the sand below which no mud was found by the borings of the engineers for the construction of the subway above referred to.

From a consideration of these and other similar data, it does not seem that the explanation quoted by Mr. Main in his paper from the report by Professor Crosby, to the effect that the various settlements noticed in the vicinity can be attributed to the presence of layers of mud or similar material in the ground upon which the structures rest, is at all adequate or convincing. It seems that either one of the following theories would more fully explain the conditions as they exist there:

First, that the materials upon which these structures which have shown settlement rest—namely, sand and a thick layer of clay—have had more or less of the water which they contain squeezed out by the unusual pressure applied there, thus allowing a gradual compression of these materials showing itself in the settlements noticed.

Second, that there has been a gradual lateral movement of the clay bed at the points where continuous and heavy pressures are applied, causing a subsidence or settlement at these points and possibly a rise in the surface at other points. A study of the borings taken here shows that the clay, while hard and firm at the bottom near bed rock, is generally quite soft and plastic so as to become mobile at the top, and may thus readily become displaced under unbalanced pressures.

Experience here seems to warrant the conclusion that buildings built upon this area, unless extraordinary means are taken

to prevent it, will be subject to a substantial amount of settlement, and that structures must be designed so that the settlement, when it occurs, shall be uniformly distributed and as little in amount as possible. It must be confessed that the conditions as they exist in this tract are such as to render the accomplishment of these designs one of great difficulty.

All engineers will watch with interest the results of the efforts by the designers of the Technology buildings to accomplish these ends.

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PAPERS AND DISCUSSIONS

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CONCRETE MATERIALS AND DESIGN OF THE NEW
BUILDINGS OF THE MASSACHUSETTS INSTITUTE
OF TECHNOLOGY, CAMBRIDGE, MASS.*

BY SANFORD E. THOMPSON,† MEMBER BOSTON SOCIETY OF CIVIL ENGINEERS.

(Presented December 19, 1917.)

THE construction of the new buildings of the Massachusetts Institute of Technology represents one of the most comprehensive schemes, for an educational institution, that has been developed in this country. Standing upon an area of 50 acres, the buildings themselves cover an area of $3\frac{1}{2}$ acres, and if placed in a straight line would extend some 2 500 ft. in length. Except the stairways, which are of structural steel, reinforced concrete was used throughout, and about 40 000 cu. yds. of concrete and 3 600 tons of steel reinforcement were required.

The main subject of this paper is the structural features of the work, more especially the engineering studies and investigations that determined them. In connection with a general description of the layout of the work, however, the more interesting construction details are taken up.

* This paper is taken in part from a paper entitled, "Design and Construction of the Massachusetts Institute of Technology Buildings," by Sanford E. Thompson, which was published by the *American Concrete Institute*, July, 1915.

† Of Thompson & Lichtner, Consulting Engineers, 136 Federal Street, Boston, Mass.

NOTE. Discussion of this paper is invited, to be received by W. L. Butcher, Editor, 715 Tremont Temple, Boston, before March 10, 1918, for publication in a subsequent issue of the JOURNAL.

ARRANGEMENT OF BUILDINGS.

The complete plan provides for over thirty buildings, with seventeen of the larger and more important grouped around a central court and so arranged and constructed as to form one continuous building.

Besides these, several academic buildings were constructed,

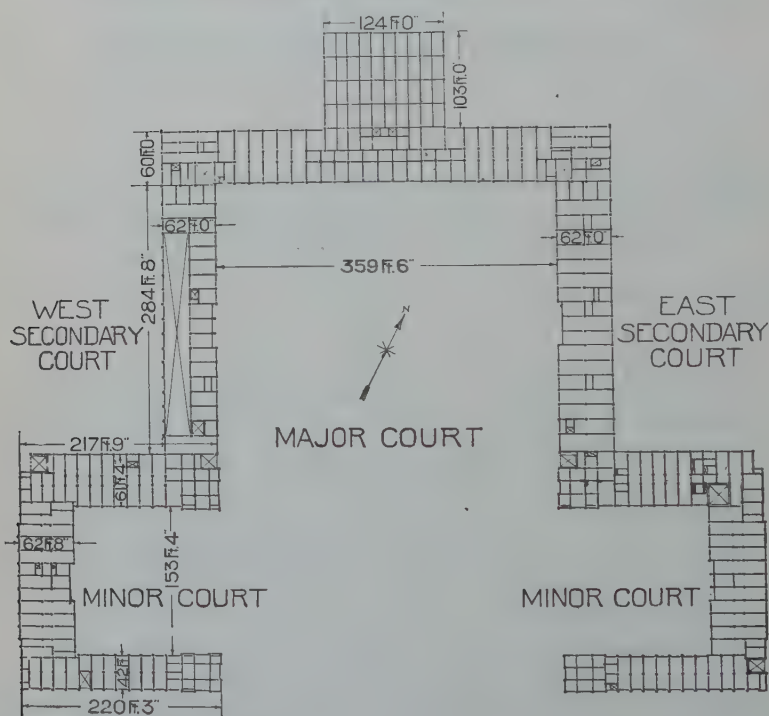


FIG. 1. ARRANGEMENT OF BUILDINGS AND FRAMING PLAN OF SECOND FLOOR.

namely, the President's house, Walker Memorial building, and dormitories. Although several of these buildings provide interesting features in design, they will not be described in this paper.

The buildings facing directly on the major and minor courts, and on the important streets, are faced with Indiana limestone. On the secondary courts and other less prominent places, pressed brick is used. See Fig. 1.

The buildings on the major court have four stories, and those on the minor courts three, but on these latter are two pavilions that run up to the fourth story, one either side of the main entrance from the street.

The Administration Building, which is in the center of the group, is five stories high and is provided with a dome.

MATERIALS, TESTS AND STRESSES.

Concrete Materials and Tests. — Although eastern Massachusetts is largely covered with glacial deposits of sand and gravel, it is by no means easy to find materials that make first-class concrete. Available sources of supply were sampled and tested before placing the contract for aggregates, and finally a new bank located about twelve miles from the site was opened and the contract made for excavation, screening and delivery.

Immediately after deciding on the aggregate, compression tests of cylinders 8 in. in diameter by 16 in. long were made, to determine the best proportions and allowable stresses. As a result, three mixes were chosen, proportions 1 : 2 : 4, for the footings and foundation walls, 1 : 1½ : 3 for the superstructure, and 1 : 1 : 2 for a few important columns that required especially high stresses.

The cement used was tested regularly at the mill with check tests at laboratory.

As sand tests are fully as important as cement tests, the inspector went to the bank at regular intervals for samples of sand which were sent to the author's laboratory for testing. The specifications required a ratio to standard sand of 70 per cent. for three days and 85 per cent. for seven days. Although this is somewhat lower than the Joint Committee recommendations, it was found that the aggregate produced concrete of the required strength and furthermore the values could be used as absolute minimums on short time tests. As a matter of fact, the tests thus far have averaged 91 per cent. for three days, 100 per cent. for seven days, and 102 per cent. for twenty-eight days, an excellent record, especially for New England sands.

Field Tests. — Two field samples of concrete were taken

regularly on each section of the job at least four times a week for test at fourteen and twenty-eight days, both samples from the same batch in order to arrive at the true relative value. Wherever possible, the samples were taken directly from place; but for beams, columns and girders, this was impossible, and samples were taken from the barrow before placing. The concrete was shoveled into two 14-qt. galvanized-iron pails, carried to the molding yard, remixed to eliminate the segregation of materials due to carrying from the work, and poured into iron molds imbedded in moist sand. The maximum and minimum temperatures were recorded every day. For mass concrete where the aggregate ran up to 2 or $2\frac{1}{4}$ ins., samples were cast in 8 by 16 in. cylinders, but for the superstructure concrete, where the largest sized aggregate was not over $1\frac{1}{2}$ ins., 6 by 6 by 12 in. prisms were used because the material for the larger size of specimen is too heavy to carry down from the upper stories. The bottom of each mold was formed by a $\frac{1}{2}$ -in. iron plate. The concrete was tamped in the mold with a 6-in. ice chopper, taking about the same precautions as were employed on the regular work. Each sample was given one tamp on each face and four tamps at 90 degrees in the center. The time of the operations of dumping into the molds and tamping never exceeded five minutes. Just previous to the initial set, the top surfaces were carefully troweled with a plasterer's trowel in order to obtain uniform bearing for the compression machine. After forty-eight hours the blocks were taken out of the sand, removed from the molds, and reburied in the moist sand where they remained until the day before testing.

Results of Tests. — The average results of these concrete compression tests were as follows:

- 1 : 2 : 4 concrete, 14 days old, 1 640 lbs.
- 1 : 2 : 4 concrete, 28 days old, 2 110 lbs.
- 1 : $1\frac{1}{2}$: 3 concrete, 14 days old, 2 008 lbs.
- 1 : $1\frac{1}{2}$: 3 concrete, 28 days old, 2 520 lbs.

Each value is an average of about 90 specimens, and the per cent. of variation from the mean runs much lower than is usual in construction work because of the care with which the samples were taken and molded.

The increase in strength from 14 to 28 days for the 1 : 2 : 4 concrete has averaged 23 per cent., and for the 1 : 1½ : 3 concrete 25 per cent. The series of laboratory tests conducted last year by the American Concrete Institute Committee on Aggregates showed an increase of 19 per cent. in this period.

The practical value of this system of sampling and testing was shown at the beginning of the job. Some of the specimens of 1 : 2 : 4 concrete gave low test. The method of handling the concrete on the job was changed at once, and subsequent specimens gave proper strengths. To be sure the concrete that had been previously placed was satisfactory, three or four samples were cut out of a solid footing three or four weeks old. After being sawed to a standard 8 by 8 by 16 in. size, they were crushed and the strengths averaged 2 200 lbs., a satisfactory figure for this portion of the structure.

Properties of Reinforcing Steel. — The cold twisted steel used for reinforcement was tested at the mill with the following results:

Yield point varied from 57 000 to 64 000 lbs. and the tensile strength from 70 000 to 81 000 lbs.

Elongation in 8 ins. was 7½ per cent. to 10 per cent., and reduction in area was 33 per cent. for the larger sizes of bars and 15 per cent. elongation with 50 per cent. reduction in area for sizes below ½ in.

The fracture was a half cup.

Bending tests for ½ in. and larger bars were 180 degrees around their own diameters, and, for bars below ½ in., 180 degrees flat.

Phosphorus ranged from .01 per cent. to .02 per cent. and sulphur from .03 per cent. to .04 per cent.

Beam Tests. — The stresses were so chosen that no cracks would be visible under ordinary working loads, and before coming to a decision on these stresses full size T-beams were made and tested.

As a result of the tests it was evident that the cracks in beams reinforced with twisted steel are less objectionable under a steel stress of 18 000 lbs. per sq. in. than cracks in beams reinforced with plain round bars under a stress of only 16 000 lbs. per sq. in. Moreover, so much of the tension was carried by the

concrete that with a theoretical stress of 18 000 lbs. the actual measured stress was only 8 000 lbs. In beams reinforced with twisted steel there were many more cracks than in beams reinforced with plain round bars, but these cracks were all small and well distributed, and therefore unobjectionable, whereas in the plain round bar beams the cracks were larger and more noticeable.

DESIGN.

Complete specifications for the structural design, following substantially the recommendations of the Joint Committee on Concrete and Reinforced Concrete, were drawn up by the author. Only special features are considered in this paper.

UNIT STRESSES.

The following standard unit stresses are based on the compressive strength of 8 by 16 in. concrete cylinders, tested in the laboratory, using the percentages for different kinds of stress recommended by the Joint Committee on Concrete and Reinforced Concrete.

Item.	1 : 2 : 4 Concrete.	1 : 1½ : 3 Concrete.	1 : 1 : 2 Concrete.
Compressive unit strength of concrete in 8" by 16 in. cylinders.....	2 000	2 500	3 300
Compressive fiber stress:			
At center of beam.....	650	810	
At support.....	750	930	
Compressive stress.....	450	560	750
Shearing unit stress.....	120	150	
Bond stress.....	80	100	
Ratio of elasticity.....	15	12	10
Steel tensile stress.....	18 000	18 000	18 000

Bending Moments. — All beams were considered as continuous. The bending moment coefficients depended upon the arrangement of the columns. In several buildings the continuous beam consisted of two long end spans and one short center span. In this case the ordinary bending moment coefficients were not sufficient and it was necessary to make special

calculations. Fig. 2 shows the typical spans, and Fig. 3 the positions of live and dead load for maximum bending moments for this type of beam. The formulas from which the bending moments are derived are rather complicated and involve integral calculus. Solutions were worked out for the different con-

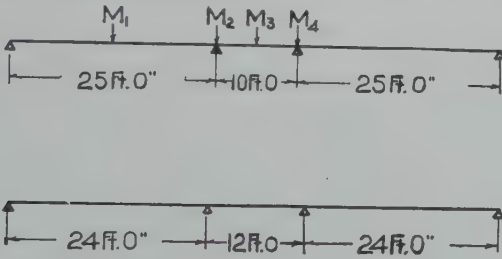


FIG. 2. TYPICAL ARRANGEMENT OF BEAM SPANS.

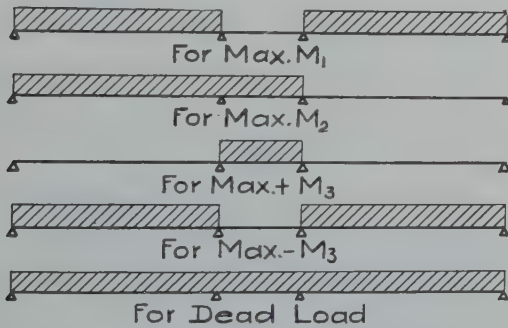


FIG. 3. DIFFERENT POSITIONS OF LIVE LOAD.

ditions, and constants obtained by use of which the general formulas were reduced to the simple ones given below.

Let

M_2 and M_4 = bending moments at center supports.

m_{x1} and m_{x3} = static bending moment at any point in the respective end span due to load in that span.

m_{x2} = static bending moment at any point in the center span due to load in that span.

s = ratio of end span to center span.

l = length of end span.

nl = length of interior span.

Then the general formulas will be as follows:

$$2M_2(l+nl)+M_4nl=-6\left[\frac{1}{l}\int_0^l m_{x_1}x dx+\frac{1}{nl}\int_0^{nl} m_{x_2}(nl-x)dx\right]$$

$$M_2nl+2M_4(nl+l)=-6\left[\frac{1}{nl}\int_0^{nl} m_{x_2}x dx+\frac{1}{l}\int_0^l m_{x_1}(l-x)dx\right]$$

These formulas were solved and constants in terms of dead and live loads derived for different ratios of short span to long span and for different positions of the load.

For design, as a result of these studies, the following simple formulas were used for the three-span continuous beam:

$$\text{End spans: Center } M_1 = \frac{1}{11}wl^2.$$

$$\text{Support } M_2 = -\frac{1}{11}wl^2.$$

$$\text{Center spans: Supports } M_2 = -\frac{1}{11}wl^2.$$

$$\text{Middle } \begin{cases} M_3 = \frac{1}{60}wl^2. \\ M_3 = -\frac{1}{18}wl^2. \end{cases}$$

In all above cases, l is the length of the end span.

In the several buildings, the arrangement consisted of one center column and two wall columns. In this case the bending moment used at the center and at the supports was $M = \frac{wl^2}{10}$.

In computing the bending moment the effect of negative moment at the wall column was considered.

In the administration buildings most of the beams extend over several spans of equal size, therefore ordinary formulas were used. Special treatment, however, was required in the long beams over the lecture rooms, the span of which was 46 ft.

SELECTION OF SPACING OF BEAMS.

The size of the panels, i. e., the arrangement of the columns, was determined by the architectural treatment. To determine

the spacing of the beams in the panel, comparative estimates were made of the relative costs of the following three cases:

Case I. Panel with no intermediate beam; slab 15 ft. 6 ins. solid concrete construction.

Case II. Panel with one intermediate beam; slab 7 ft. 9 ins.

Case III. Panel with no intermediate beam; slab 15 ft. 6 ins. with tile.

The panel with no intermediate beam, slab 15 ft. 6 ins. solid concrete construction, proved the most economical. The interesting result of the estimate was that the cost of forms and not the cost of materials was the determining factor. For instance, in the selected type, the cost of materials was \$134 per panel, while in the type with one intermediate beam the cost of materials was only \$130. However, the cost of forms in the first case was \$36.25 against \$48.27 in the second case, so that the total cost of the accepted type was \$170.25 against \$178.27 of the second type. The cost of the hollow tile construction was more expensive than either of the two concrete types, mainly due to the cost of the tile.

Attention should be called, of course, to the fact that with prevailing high prices of materials, the relative economy of the various types may be different.

In computing cost of forms the book on "Concrete Costs," by Taylor & Thompson, was found very useful.

ADMINISTRATION BUILDING.

The design of the Administration Building presents several features of particular interest. The building is five stories high, and the height from the basement level to the highest point of the dome is around 151 ft.

The building, besides offices and regular class rooms, has two lecture rooms, one on the second floor and one on the fourth floor. The dimensions of the lecture rooms are 72 ft. by 72 ft. The lecture room on the second floor, which is two stories high, is provided with seats amphitheatrically arranged. These rooms have only two interior columns. The girders supporting the floors over the lecture rooms were 48 ft. long, and their di-

mensions 24 ins. by 78 ins. The girders were of special section. The beams supported by these girders were 24 ft. long, spaced 12 ft. on centers.

The fifth floor of the Administration Building, which is used for the library, is of circular shape with a radius of 60 ft. This portion, besides the wall columns arranged in a circle, has eight interior columns which extend to the top of the dome. The wall columns and the wall of the circular portion are supported by a ring girder, which in turn rests on 24-in. by 60-in. girders.

The open space in the library is circular, and is of 72 ft. diameter.

The roof of this circular portion of the Administration Building has the appearance of a dome, the diameter of which is 108 ft.

Originally it was intended to construct the dome of steel ribs; also, a gustovina tile dome of a diameter of 108 ft. was considered. Upon further investigation, however, it was decided to carry the eight interior columns, mentioned before, to the top of the dome, thereby reducing its span from 108 ft. to around 72 ft.

The roof, therefore, is composed of the dome proper of 72 ft. diameter, and a slanting beam and slab construction in which the beams are arranged radially, giving the outside effect of a dome. With this arrangement, it was found possible to construct the entire dome of reinforced concrete.

The slab composing the dome proper was 5 ins. thick on the top, and 8 ins. thick at the haunch. A reinforced concrete ring of suitable dimensions was provided at the haunch, to take the thrust of the dome. This ring rested at the top of the eight columns. The remaining portion of the dome consisted of beam and slab construction, where the beams and slabs were inclined to give the circular effect. The dome slab was reinforced with radial bars and concentric rings.

BEAM REINFORCEMENT.

Beams are all continuous, and negative moments were provided for in the design. The location of bends in steel was

always selected so as to leave as much steel as was required for both tension and compression stresses. As a rule, about three eighths of the shear was considered to be carried by the bent

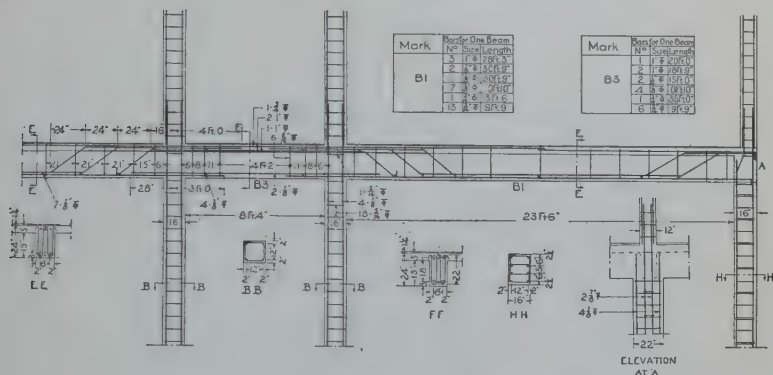


FIG. 4. TYPICAL BEAM AND COLUMN ON MINOR COURT.

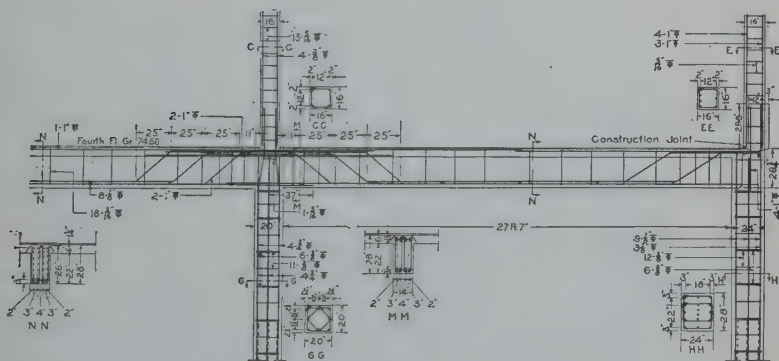


FIG. 5. TYPICAL BEAM AND COLUMN ON MAJOR COURT.

bars in accordance with recent tests. Typical reinforced beams and columns are shown in Fig. 4 and Fig. 5, and typical cross sections of the buildings are shown in Figs. 6 and 7.

Unusually long beams of special design were used in the construction of the pavilions in Buildings 1, 2, 5 and 6, where it was desirable to have class rooms over 40 ft. sq. with no inside columns. Four beams running at right angles to each other,

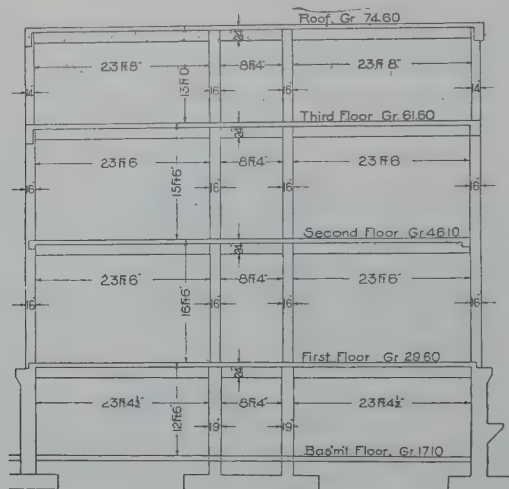


FIG. 6. CROSS SECTION OF BUILDING ON MINOR COURT.

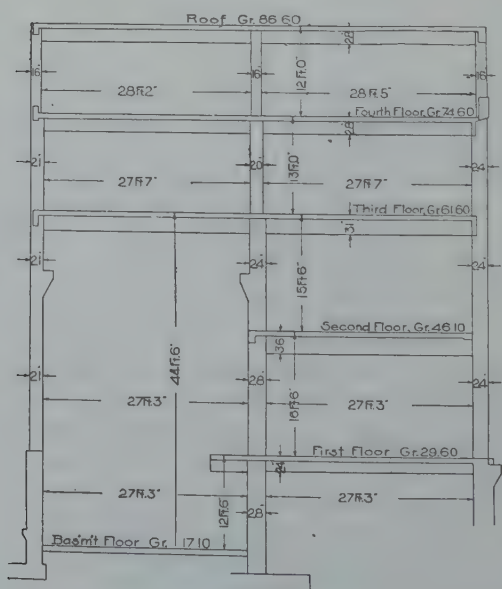


FIG. 7. CROSS SECTION OF BUILDING ON MAJOR COURT.

two in each direction, divided the floor into nine nearly square panels. The beams were 18 by 36 ins., and the reinforcement consisted of six 1-in. and six $\frac{7}{8}$ -in. bars at the center and five 1-in. and two $\frac{7}{8}$ -in. bars at the ends. All bars were square twisted steel. The pavilion roof beams are also unusually large because of the architectural treatment.

Interior Columns. — All interior columns are square, and reinforced with vertical steel only. Bands were placed 12 ins. on centers to keep the steel in position during construction. The eccentricity of interior columns was neglected because the short beams were designed with sufficient steel to prevent bending in the column.

Wall Columns. — In the design of wall columns account was taken of the bending moment produced by the rigidity of the connection between beam and column. The eccentricity in this case was computed carefully and enough steel provided to take care of it. To provide for the bending moment in the top story, however, it would have been necessary to use larger columns than the architectural treatment would permit. Therefore, to prevent a bending moment in the columns, it was decided to make a joint between the columns and roof beams and not to carry the outside reinforcement through the beam. Columns were then designed for the superimposed load placed on the column with an eccentricity of 0.4 the width of the column.

Slabs. — The floor slabs, which are continuous, were designed for a bending moment $\frac{wl^2}{12}$ at the support and the same moment in the center of the span. To prevent cracks in the building as far as possible, short bars were placed on the top over each girder whenever the main slab reinforcement ran parallel to the girder, and, in fact, wherever there was danger of negative stresses.

Walls. — Retaining walls were cast monolithic with the wall columns except on the outside wall. In the minor and major courts separate retaining walls were built about 3 ft. 9 ins. from the building line.

Floor Surfaces. — Special studies of floors were made and several experimental slabs were built in order to find the method

of placing that would result in minimum dusting and a satisfactory bond with the slab. The scum which always rises on the surface of base concrete was removed by brushing the slab with a wire brush just as it begins to harden. The tests showed that dusting is best prevented by using, with granolithic, coarse sand containing scarcely any dust and mixed with broken stone ranging from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in size and mixed in a stiff mortar in the proportions $1 : \frac{3}{4} : 1\frac{1}{4}$. As a result of laboratory tests, stiff neat cement brushed into the set concrete was selected as the best bonding material between the base concrete and granolithic.

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